CALIFORNIA
DEPARTMENT OF TRANSPORTATION

manual change transmittal

<table>
<tr><td>TITLE
DIVISION OF DESIGN
HIGHWAY DESIGN MANUAL

SIXTH EDITION – CHANGE 10/04/10</td><td>APPROVED BY

TERRY L. ABBOTT, Chief</td><td>NO.

Date Issued:
10/01/10

Page 1 of 4</td></tr>
<tr><td>SUBJECT AREA
Forward; Table of Contents; List of Figures; List of Tables; Chapters: 100, 200, 300, 500, 800-840 and 870</td><td colspan="2">ISSUING UNIT

DIVISION OF DESIGN</td></tr>
<tr><td>SUPERCEDES

SEE BELOW FOR SPECIFIC PAGE NUMBERS</td><td colspan="2">DISTRIBUTION
ALL HOLDERS OF THE 6TH EDITION, HIGHWAY DESIGN MANUAL</td></tr>
</table>

The California Department of Transportation has identified several typographical errors in the Sixth Edition, Highway Design Manual (HDM). Typographical corrections to Chapters 100, 200, 300, and 500 are included in this manual change transmittal. The Table of Contents; List of Figures; and List of Tables have also been revised to reflect updated highway drainage design guidance provided in Chapters: 800-840, and 870. The errata and changes are described in the summary below with change-sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. These revisions and changes are effective October 4, 2010, and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

HDM Holders are encouraged to use the most recent version of the HDM available on-line at the above website. Should a HDM Holder choose to maintain a paper copy, the Holder is responsible for keeping their paper copy up to date and current. Using the latest version available on-line will ensure proper reference to the latest design standards and guidance. If you would like to be notified automatically of any significant changes or updates to the HDM, go to http://www.dot.ca.gov/hq/oppd/hdm/hdmlist.htm.

A summary of the most significant revisions are as follows:

Forward	**Metric Basics** Upper left corner of page, the incorrect page number, 500-2, was deleted.
Index 110.12	**Tunnel Safety Orders, Page 100-28** Updated to reflect California Code of Regulations, Title 8, Subchapter 20, change in procedures per the Department of Industrial Relations, Division of Occupational Safety and Health (Cal-OSHA), Mining and Tunneling Unit.
Figure 201.4	**Stopping Sight Distance on Crest Vertical Curves, Page 200-5** The border around the graph at the bottom of the page was darkened. The plot of the S=L line was corrected.
Figure 201.5	**Stopping Sight Distance on Sag Vertical Curves, Page 200-6** The border around the graph at the bottom of the page was darkened.

Figure 201.6	**Stopping Sight Distance on Horizontal Curves, Page 200-7** The border around the graph at the bottom of the page was darkened. Design speeds in mph were deleted from the plotted lines and correctly provided along the top axis of the graph corresponding to stopping sight distance. The label for the y-axis of the graph was corrected to present clear distance to obstruction (*m*)- (feet) for consistency with the description at the top of the figure.
Figure 201.7	**Decision Sight Distance on Crest Vertical Curves, Page 200-8** The border around the graph at the bottom of the page was darkened. The sight distance value, when V=30 mph and K=170 was corrected from 475 ft to 525 ft.
Figure 202.5B	**Superelevation Transition Terms & Definitions, Page 200-14** The superelevation runoff point where 2/3L and 1/3L meet at the BC or EC was corrected on the figure to occur between positions 5 and 6 for consistency with Figure 202.5A.
Index 204.8	**Grade Line of Structures, Page 200-22** Page breaks were corrected, beginning with this page.
Index 206.3(2)	**Pavement Reductions, Page 200-28** The reference where the standard taper for a ramp merge into through traffic was corrected from Figure 504.2L to 504.3L.
Figure 309.2	**Department of Defense Rural and Single Interstate Routes, Page 300-25** The term "vicinity" was misspelled and corrected.
Table 504.3A	**Ramp Widening for Trucks, Page 500-15** Lane width for a ramp radius of less than 150 ft was corrected from 17 ft to 18 ft.
Index 504.3(3)	**Ramps, Page 500-20** Fifth paragraph, for corner sight distance, the driver's eye is assumed to be located 10 ft from the edge of shoulder, not 8 ft as stated. This correction is consistent with Figure 504.3J.
Index 801.3	**Drainage Standards, Page 800-1** Font and general page layout corrections were made to this index.
Chapter 800	**General Aspects, Page 800-1** The previous titles of Division of Structures and Structures Design were corrected with the current title Division of Engineering Services throughout the chapter.
Figure 804.7A	**Technical Information for Location Hydraulic Study, Page 800-11** Separate yes and no entries were added to indicate availability of either NFIP maps or studies, since one may be available without the other.
Index 805.4	**Unusual Hydraulic Structures, Page 800-15** FHWA Headquarters Office of Bridge Technology approval was added for hydraulic structure projects on the interstate system involving unusual stream stability countermeasures or unique design techniques.

Index 806.2	**Drainage Terms, Page 800-16** Various terms were added and defined.
Table 816.6A	**Roughness Coefficients For Sheet Flow, Page 810-11** Footnote (1) was corrected to express woods cover height in U.S. Customary (English) units rather than metric.
Index 819.1	**Introduction, Page 810-15** Second paragraph was added to provide reference to the expanded guidance on estimating design discharge via region-specific analysis.
Index 819.2(2)	**Empirical Methods, Page 810-16** USGS regression equations based on a 1994 study have been supplemented with more recent (2007) study data for California desert regions. Figure 819.2D has been eliminated and Figure 819.7A and Table 819.7A have been inserted to reflect the new data.
Table 819.5A	**Summary of Methods for Estimating Design Discharge, Page 810-22** Replaced the USGS Open-File Report Method 93-419 with the Improved Highway Design Methods for Desert Storms. Also removed the NRCS unit Hydrograph method and added the SCS Unit Hydrograph and S-Graph Unit Hydrograph.
Index 819.7	**Region-Specific Analysis, Page 810-23** Added new guidance on region-specific analysis for estimating design discharge, incorporating discussion of desert storm hydrology.
Index 821.3	**Selection of Design Flood, Page 820-2** The guidance was shown to exclude culvert structures that meet the definition of a bridge. The effects of bedload and debris were added to determining freeboard during design flood.
Index 834.2(1)	**Median Drainage, Page 830-6** Reference to further median drainage considerations in Index 305.3(9) was deleted as this reference was previously changed.
Index 837.3(4)	**Location and Spacing, Page 830-15** The recommended spacing of inlets in series was corrected to indicate 20 ft rather than 6 m.
Index 841.5	**Category of System, Page 840-2** First bullet, the reference to Topic 606 was corrected to indicate Chapter 650, Pavement Drainage.
Index 871.2	**Design Philosophy, Page 870-2** The definition of expendability was revised.
Index 872.3	**Site Consideration, Page 870-4** Minor revision to some of the bullets under issues that may lead to common protection failures.

Table 872.2 **Failure Modes and Effects Analysis for Riprap Revetment, Page 870-6**
New table added.

Index 873.3(2)(a)(1)(d) **Armor Protection, Page 870-23**
Rock slope protection fabric reference to new Gravel Filter Index 873.3(2)(a)(1)(e) was added.

Index 873.3(2)(a)(2) **Armor Protection, Page 870-25**
The eleventh bullet was added to well designed streambank rock slope protection considerations.

Enclosures available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm.

FOREWORD

Purpose

This manual was prepared by the Division of Design for Project Delivery. The manual establishes uniform policies and procedures to carry out the highway design functions of the California Department of Transportation (Department). It is neither intended as, nor does it establish, a legal standard for these functions.

The policies established herein are for the information and guidance of the officers and employees of the Department, as well as external agencies who use or choose to adopt this guidance.

Many of the instructions given herein are subject to amendment as conditions and experience seem to warrant. Special situations may call for variation from policies and procedures, subject to Division of Design approval, or such other approval as may be specifically provided for in the text.

It is not intended that any standard of conduct or duty toward the public shall be created or imposed by the publication of the manual. Statements as to the duties and responsibilities of any given classification of officers or employees mentioned herein refer solely to duties or responsibilities owed by these in such classification to their superiors. However, in their official contacts, each employee should recognize the necessity for good relations with the public.

Scope

This manual is not a textbook or a substitute for engineering knowledge, experience, or judgment. It includes techniques as well as graphs and tables not ordinarily found in textbooks. These are intended as aids in the quick solutions of field and office problems. Except for new developments, no attempt is made to detail basic engineering techniques; for these, standard textbooks should be used.

Form

The loose-leaf form was chosen because it facilitates change and expansion. New instructions or updates will be issued as sheets in the format of this manual made available on-line on the Department Design website: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. The new instructions or updates may consist of additional sheets or new sheets to be substituted for those superseded. Users of this manual are encouraged to utilize the most recent version available on-line on the Department Design website.

Organization of the Manual

A decimal numbering system is used which permits identification by chapter, topic, and index, each of which is a subdivision of the preceding classification. For example:

Chapter 40 Federal-Aid

Topic 42 Federal-Aid System

Index 42.2 Interstate

The upper corner of each page shows the page number and the date of issue.

Use the Table of Contents

The Table of Contents gives the index number and page number for each topical paragraph together with corresponding dates of issue. If the holder of the manual chooses to maintain a paper copy, the holder is responsible for keeping the paper copy up to date and current. Revised Table of Contents will be issued on the Department Design website as the need arises.

Use of the English and Metric Editions of the Highway Design Manual

This Sixth Edition of the Highway Design Manual is in U.S. Customary (English) units. All projects designed and constructed in English units shall follow the standards in this manual per the instructions contained in Index 82.5, "Effective Date for Implementing Revisions to Design Standards".

The Metric standards contained in the Fifth Edition of the Highway Design Manual, and related publications, are to continue to be used if the specific project was granted an exception to be delivered in Metric units. See Department memorandum dated June 16, 2006, signed by Richard D. Land, entitled "Declaration of Units of Measure – "Metric" or "English" Project."

Metric Basics

Measurable Attribute - Basic Units	Unit	Expression
Length	meter	m
Mass	kilogram	kg
Luminous intensity	candela	cd
Time	second	s
Time	hour	h
Electric current	ampere	A
Thermodynamic temperature	Kelvin	K
Amount of substance	mole	mol
Volume of liquid	liter	L
Measurable Attribute - Special Names	**Unit**	**Expression**
Frequency of a periodic phenomenon	hertz	Hz (1/s)
Force	newton	N ($kg \cdot m/s^2$)
Energy/work/quantity of heat	joule	J($N \cdot m$)
Power	watt	W (J/s)
Pressure/stress	pascal	Pa (N/m^2)
Celcius temperature	Celsius	°C
Quantity of electricity/electrical charge	coulomb	C
Electric potential	volt	V
Electric resistance	ohm	Ω
Luminous flux	lumen	lm
Luminance	lux	lx (lm/m^2) or (cd/m^2)
Measurable Attribute - Derived Units	**Unit**	**Expression**
Acceleration	meter per second squared	m/s^2
Area	square meter	m^2
Area	hectare	ha (10 000 m^2)
Density/mass	kilogram per cubic meter	kg/m^3
Volume	cubic meters	m^3
Velocity	meter per second	m/s
Mass	tonne	tonne (1000 kg)

Multiplication Factors	Prefix	Symbol	Pronunciations
1 000 000 000 = 10^9	giga	G	jig' a (*i* as in j*i*g, *a* as in *a*-bout)
1 000 000 = 10^6	mega	M	as in *mega*-phone
1000 = 10^3	kilo	k	kill' oh
100 = 10^2	*hecto	h	heck' toe
10 = 10^1	*deko	da	deck' a (*a* as in *a*-bout)
0.1 = 10^{-1}	*deci	d	as in *deci*-mal
0.01 = 10^{-2}	*centi	c	as in *centi*-pede
0.001 = 10^{-3}	milli	m	as in *mili*-tary
0.000 001 = 10^{-6}	micro	μ	as in *micro*-phone
0.000 000 001 = 10^{-9}	nano	n	nan' oh (*an* as in *an*t)

* to be avoided where possible

Common Conversion Factors to Metric

Class	Multiply:	By:	To Get:
Area	ft^2	0.0929	m^2
	yd^2	0.8361	m^2
	mi^2	2.590	km^2
	acre	0.404 69	ha
Length	ft	0.3048	m
	in	25.4	mm
	mi	1.6093	km
	yd	0.9144	m
Volume	ft^3	0.0283	m^3
	gal	3.785	L *
	fl oz	29.574	mL *
	yd^3	0.7646	m^3
	acre ft	1233.49	m^3
Mass	oz	28.35	g
	lb	0.4536	kg
	kip (1,000 lb)	0.4536	tonne (1000 kg)
	short ton (2,000 lb)	907.2	kg
	short ton	0.9072	tonne (1000 kg)
Density	lb/yd^3	0.5933	kg/m^3
	lb/ft^3	16.0185	kg/m^3
Pressure	psi	6894.8	Pa
	ksi	6.8948	MPa (N/mm^2)
	lb_f/ft^2	47.88	Pa
Velocity	ft/s	0.3048	m/s
	mph	0.4470	m/s
	mph	1.6093	km/h
Temp	°F	$t_{°C} = (t_{°F}-32)/1.8$	°C
Light	footcandle (or) lumen/ft^2	10.7639	lux (lx) (or) lumen/m^2

* Use Capital "L" for liter to eliminate confusion with the numeral "1"

Land Surveying Conversion Factors

Class	Multiply :	By:	To Get
Area	acre	4046.87261	m^2
	acre	0.404 69	ha (10 000 m^2)
Length	ft	1200/3937**	m

** Exact, by definition of the US Survey foot, Section 8810, State of California Public Resources Code

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CHAPTER 860 - OPEN CHANNELS

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(3) *Use of Flexible Pavement Grindings, Chunks and Pieces.* When constructing transportation facilities, the Department frequently uses asphalt in mixed or combined materials such as flexible pavement. The Department also uses recycled flexible grindings and chunks. There is a potential for these materials to reach the waters of the State through erosion or inappropriate placement during construction. Section 5650 of the Fish and Game Code states that it is unlawful to deposit asphalt, other petroleum products, or any material deleterious to fish, plant life, or bird life where they can pass into the waters of the State. In addition, Section 1601 of the Fish and Game Code requires notification to the California Department of Fish and Game (DFG) prior to construction of a project that will result in the disposal or deposition of debris, waste, or other material containing crumbled, flaked, or ground pavement where it can pass into any river, stream, or lake designated by the DFG.

The first step is to determine whether there are waters of the State in proximity to the project that could be affected by the reuse of flexible pavement. Waters of the State include: (1) perennial rivers, streams, or lakes that flow or contain water continuously for all or most of the year; or (2) intermittent lakes that contain water from time to time or intermittent rivers or streams that flow from time to time, stopping and starting at intervals, and may disappear and reappear. Ephemeral streams, which are generally exempt under provisions developed by the Department and DFG, are those that flow only in direct response to rainfall.

The reuse of flexible pavement grindings will normally be consistent with the Fish and Game Code and not require a 1601 Agreement when these materials are placed where they cannot enter the waters of the State. However, there are no set rules as to distances and circumstances applicable to the placement of asphaltic materials adjacent to waters of the State. Placement decisions must be made on case-by-case basis, so that such materials will be placed far enough away from the waters of the State to prevent weather (erosion) or maintenance operations from dislodging the material into State waters. Site-specific factors (i.e., steep slopes) should be given special care. Generally, when flexible pavement grindings are being considered for placement where there is a potential for this material to enter a water body, DFG should be notified to assist in determining whether a 1601 Agreement is appropriate. DFG may require mitigation strategies to prevent the materials from entering the Waters of the State. When in doubt, it is recommended that the DFG be notified.

If there is the potential for reused flexible materials to reach waters of the State through erosion or other means during construction, such work would normally require a 1601 Agreement. Depending on the circumstances, the following mitigation measures should be taken to prevent flexible grindings from entering water bodies:

- The reuse of flexible pavement grindings as fill material and shoulder backing must conform to the California Department of Transportation (Department) Standard Specifications, applicable manuals of instruction, contract provisions, and the MOU described below.
- Flexible chunks and pieces in embankment must be placed above the water table and covered by at least one foot of material.

A Memorandum of Understanding (MOU) dated January 12, 1993, outlines the interim agreement between the DFG and the Department regarding the use of asphaltic materials. This MOU provides a working agreement to facilitate the Department's continued use of asphaltic materials and avoid potential conflicts with the Fish and Game Code by describing conditions where use of asphalt road construction material by the Department would not conflict with the Fish and Game Code.

Specific Understandings contained in the MOU are:

- Asphalt Use in Embankments

  The Department may use flexible pavement chunks and pieces in embankments when these materials are placed where they will not enter the waters of the State.

- Use of flexible pavement grindings as Shoulder Backing

  The Department may use flexible pavement grindings as shoulder backing when these materials are placed where they will not enter the waters of the State.

- Streambed Alteration Agreements

  The Department will notify the DFG pursuant to Section 1601 of the Fish and Game Code when a project involving the use of asphaltic materials or crumbled, flaked, or ground pavement will alter or result in the deposition of pavement material into a river, stream, or lake designated by the DFG. When the proposed activity incorporates the agreements reached under Section 1601 of the Fish and Game Code, and is consistent with Section 5650 of the Fish and Game Code and this MOU, the DFG will agree to the use of these materials.

There may be circumstances where agreement between the DFG and the Department cannot be reached. Should the two agencies reach an impasse, the agencies enter into a binding arbitration process outlined in Section 1601 of the Fish and Game Code. However, keep in mind that this arbitration process does not exempt the Department from complying with the provisions of the Fish and Game Code. Also it should be noted that this process is time consuming, requiring as much as 72 days or more to complete. Negotiations over the placement of flexible pavement grindings, chunks, and pieces are to take place at the District level as part of the 1601 Agreement process.

110.12 Tunnel Safety Orders

Projects and work activities that include human entry into regulatory defined tunnels or shafts to conduct construction activities must address the requirements of the California Code of Regulations (CCR), Title 8, Subchapter 20 – Tunnel Safety Orders (TSO). Activities that can be considered of a maintenance nature, such as cleaning of sediment and debris from culverts or inspection (either condition inspection for design purposes or inspection as a part of construction close-out) of tunnels, shafts or other underground facilities are not affected by these regulations.

TSO requires the Department, as owner of the facility, to request the Department of Industrial Relations, Division of Occupational Safety and Health (Cal-OSHA), Mining and Tunneling Unit, to review and classify tunnels and shafts for the potential presence of flammable gas and vapors prior to bidding. The intent of the TSO regulations are to protect workers from possible injury due to exposure to hazardous conditions. Failure to comply is punishable by fine. The complete TSO regulations are available at the following website: (http://www.dir.ca.gov/title8/sub20.html), with Sections 8403 and 8422 containing information most applicable to project design.

The TSO regulations require classification whenever there is human entry into a facility defined as a tunnel or entry into, or very near the entrance of, a shaft. Some of the common types of activities where human entry is likely and that will typically require classification include:

- Pipe jacking or boring operations, 30 inches or greater in diameter
- Large diameter pile construction, as described in the following text
- Pipeline work connected to a tunnel
- Well construction
- Cofferdam excavations
- Deep structure footings/shafts/casings, as described in the following text

The regulations apply to underground structures of 30 inches or greater diameter or shaft excavations of 20 feet or more in depth. Since a shaft is defined as any excavation with a depth at least twice its greatest cross section, the regulations will apply to some structure footing or cofferdam excavations.

Cut and cover operations (typical of most pipe, junction structure and underground vault construction) do not fall under the TSO regulations. Connecting new pipe to tunnels does fall under the TSO regulations unless the existing pipe system is physically separated by a bulkhead to prevent entry into the buried portion. Designers must either incorporate requirements for such separation of facilities into the PS&E or they must obtain the required classification from Cal-OSHA. For any project that requires classification, the designer must include information in the bid package that will alert the bidder to the specific location and classification that Cal-OSHA has provided.

The TSO regulations should be viewed as being in addition to, and not excluding, other requirements as may apply to contractor or Department personnel covered in the Construction Safety Orders (see CCR, Title 8, Subchapter 4, Article 6 at http://www.dir.ca.gov/title8/sub4.html), safety and health procedures for confined spaces (see Chapter 14 of the Caltrans Safety Manual), or any other regulations that may apply to such work.

Prior to PS&E submittal on a project that includes any work defined in CCR Section 8403, a written request must be submitted for classification to the appropriate Mining and Tunneling (M&T) Unit office. Each M&T Unit office covers specific counties as shown on Figure 110.12. Classification must be obtained individually for each separate location on a project. For emergency projects or other short lead-time work, it is recommended that the appropriate M&T Unit office be contacted as soon as possible to discuss means of obtaining classification prior to the start of construction activities.

The request must include all pertinent and necessary data to allow the M&T Unit to classify the situation. The data specified under paragraph (a) of Section 8422 (complete text of Section 8422 reprinted below) is typical of new construction projects. The appropriate M&T Unit office should be contacted for advice if there is any question regarding data to submit.

In many instances it may not be known during design if there will be human entry into facility types that would meet the definition of a tunnel or shaft. If there is any anticipation that such entry is likely to occur, classification should be requested. As permit acquisition is typically the responsibility of the District, it is imperative that there be close coordination between District and Structures Design staff regarding the inclusion of any facilities in the structures PS&E that could be defined as a tunnel or shaft and have potential for human entry. The following text is taken directly from Section 8422:

8422 Tunnel Classifications

(a) When the preliminary investigation of a tunnel project is conducted, the owner or agency proposing the construction of the tunnel shall submit the geological information to the Division for review and classification relative to flammable gas or vapors. The preliminary classification shall be obtained from the Division prior to bidding and in all cases prior to actual underground construction. In order to make the evaluation, the following will be required:

(1) Plans and specifications;

(2) Geological report;

(3) Test bore hole and soil analysis log along the tunnel alignment;

(4) Proximity and identity of existing utilities and abandoned underground tanks.

(5) Recommendation from owner, agency, lessee, or their agent relative to the possibility of encountering flammable gas or vapors;

(6) The Division may require additional drill hole or other geologic data prior to making gas classifications.

(b) The Division shall classify all tunnels or portions of tunnels into one of the following classifications:

(1) Nongassy, which classification shall be applied to tunnels where there is little likelihood of encountering gas during the construction of the tunnel.

(2) Potentially gassy, which classification shall be applied to tunnels where there is a possibility flammable gas or hydrocarbons will be encountered.

(3) Gassy, which classification shall be applied to tunnels where it is likely gas will be encountered or if a concentration greater than 5 percent of the LEL of:

(A) flammable gas has been detected not less than 12 inches from any surface in any open workings with normal ventilation.

(B) flammable petroleum vapors that have been detected not less than 3 inches from any surface in any open workings with normal ventilation.

(4) Extrahazardous, which classification shall be applied to tunnels when the Division finds that there is a serious danger to the safety of employees and:

Flammable gas or petroleum vapor emanating from the strata has been ignited in the tunnel; or

(A) A concentration of 20 percent of the LEL of flammable gas has been detected not less than 12 inches from any surface in any open working with normal ventilation; or

(B) A concentration of 20 percent of LEL petroleum vapors has been detected not less than three inches from any surface in any open workings with normal ventilation.

(c) A notice of the classification and any special orders, rules, special conditions, or regulations to be used shall be prominently posted at the tunnel job site, and all personnel shall be informed of the classification.

(d) The Division shall classify or reclassify any tunnel as gassy or extrahazardous if the preliminary investigation or past experience indicates that any gas or petroleum vapors in hazardous concentrations is likely to be encountered in such tunnel or if the tunnel is connected to a gassy or extrahazardous excavation and may expose employees to a reasonable likelihood of danger.

(e) For the purpose of reclassification and to ensure a proper application of classification, the Division shall be notified immediately if a gas or petroleum vapor exceeds any one of the individual classification limits described in subsection (b) above. No underground works shall advance until reclassification has been made.

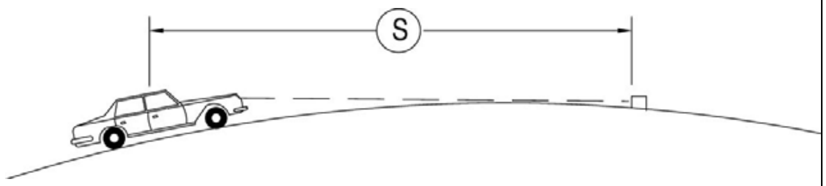
(1) A request for declassification may be submitted in writing to the Division by the employer and/or owner's designated agent whenever either of the following conditions occur:

(A) The underground excavation has been completed and/or isolated from the ventilation system and/or other excavations underway, or

(B) The identification of any specific changes and/or conditions that have occurred subsequent to the initial classification criteria such as geological information, bore hole sampling results, underground tanks or utilities, ventilation system, air quality records, and/or evidence of no intrusions of explosive gas or vapor into the underground atmosphere.

NOTE: The Division shall respond within 10 working days for any such request. Also, the Division may request additional information and/or require specific conditions in order to work under a lower level of classification.

Figure 201.4
Stopping Sight Distance on Crest Vertical Curves



Drivers eye height is 3 ½ feet.
Object height is ½-foot.

L = Curve Length (feet)

A = Algebraic Grade Difference (%)

S = Sight Distance (feet)

V = Design Speed for "S" in mph

K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Notes:

- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve

When S > L	**When S < L**
L = 2S – 1329/A	**L = AS^2 /1329**

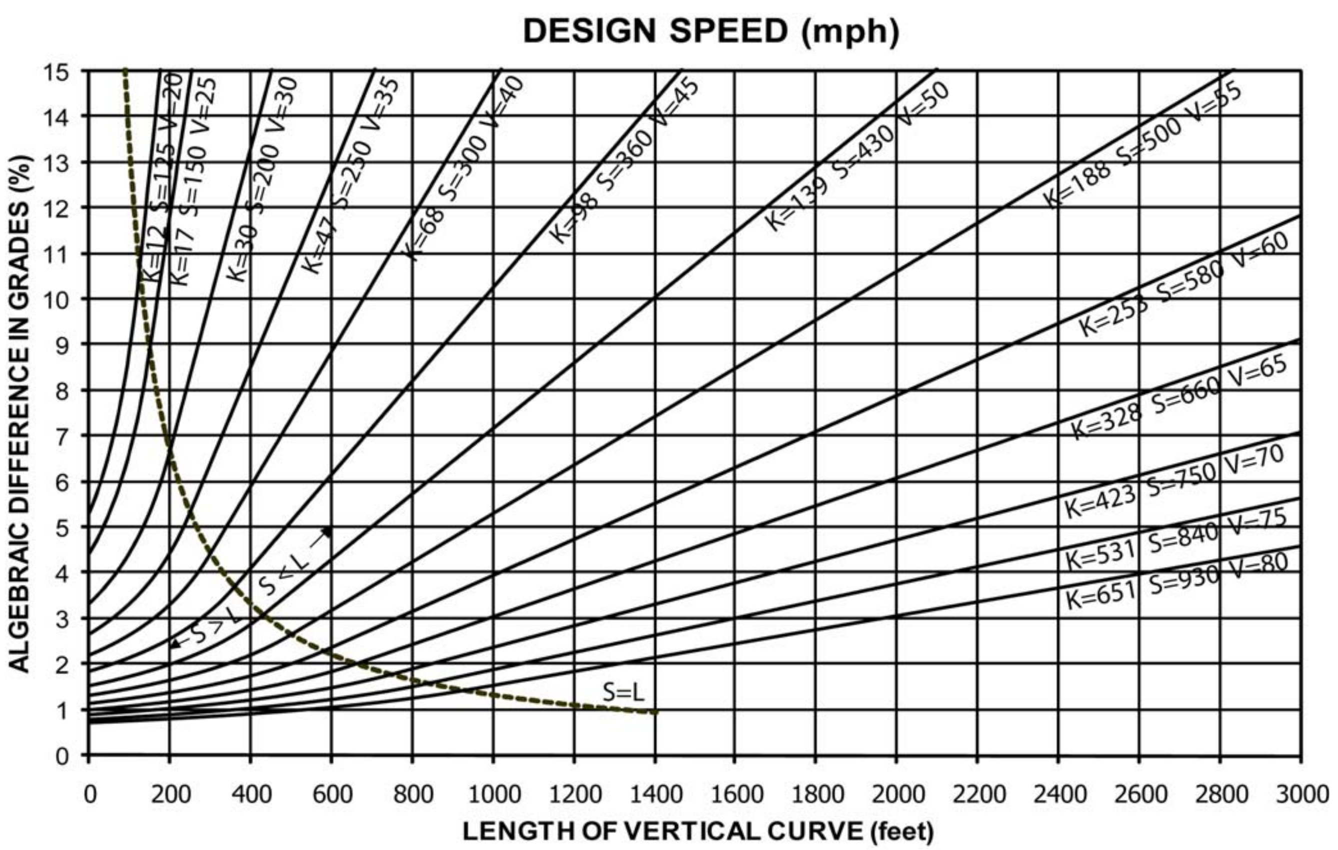
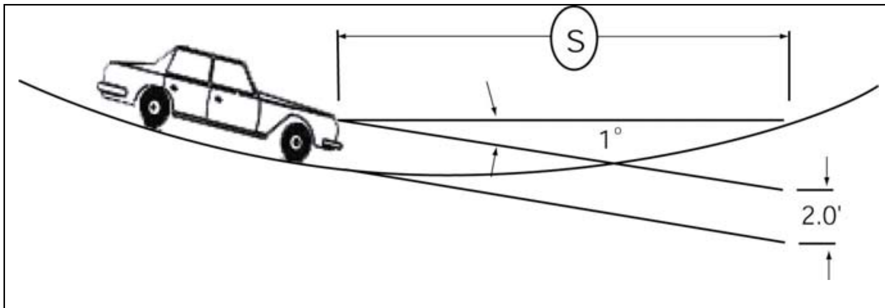


Figure 201.5
Stopping Sight Distance on Sag Vertical Curves



L = Curve Length (feet)
A = Algebraic Grade Difference (%)
S = Sight Distance (feet)
V = Design Speed for "S" in mph
K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Notes:

- For sustained downgrades, see Index 201.3.
- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve.

When S > L	**When S < L**
$L = 2S - (400 + 3.5S)/A$	$L = AS^2/(400 + 3.5S)$

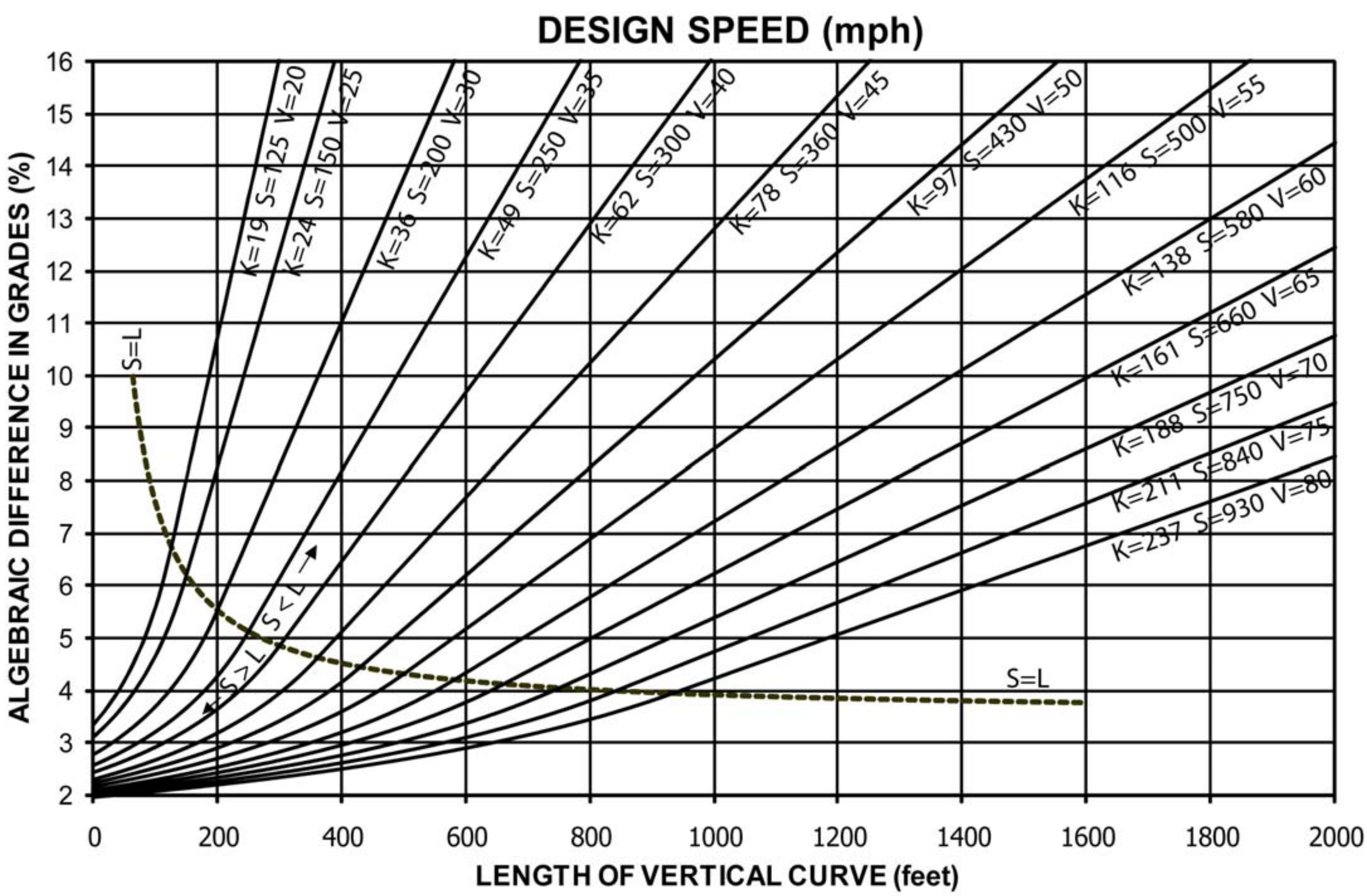
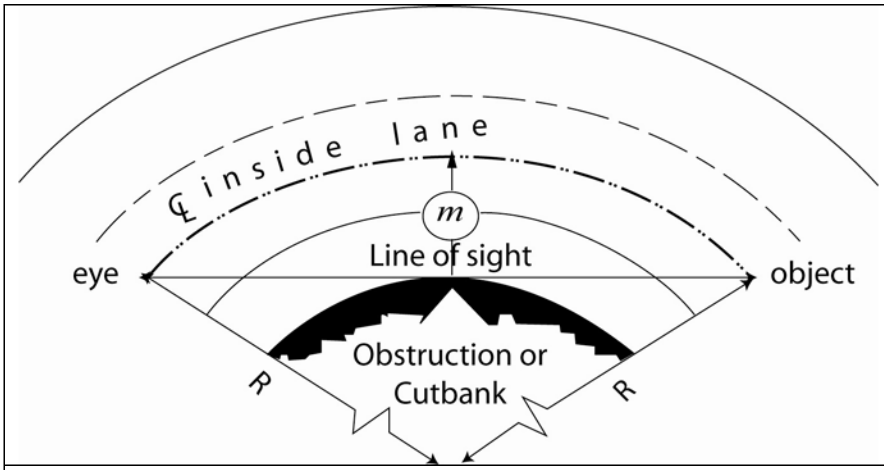


Figure 201.6
Stopping Sight Distance on Horizontal Curves



Line of sight is 2.0 feet above the centerline inside lane at point of obstruction.

R = Radius of the centerline of the lane nearest the obstruction (feet).

S = Sight Distance (feet)

V = Design Speed for "S" in mph

m = Clear distance from centerline of the lane nearest the obstruction (feet).

Notes:

- For sustained downgrades, see Index 201.3.
- Formulas apply only when "S" is equal to or less than length of curve.
- Angles in formulas are expressed in degrees.

$$m = R\left[1 - \mathrm{COS}\left(\frac{28.65S}{R}\right)\right]$$

$$S = \frac{R}{28.65}\left[\mathrm{COS}^{-1}\left(\frac{R - m}{R}\right)\right]$$

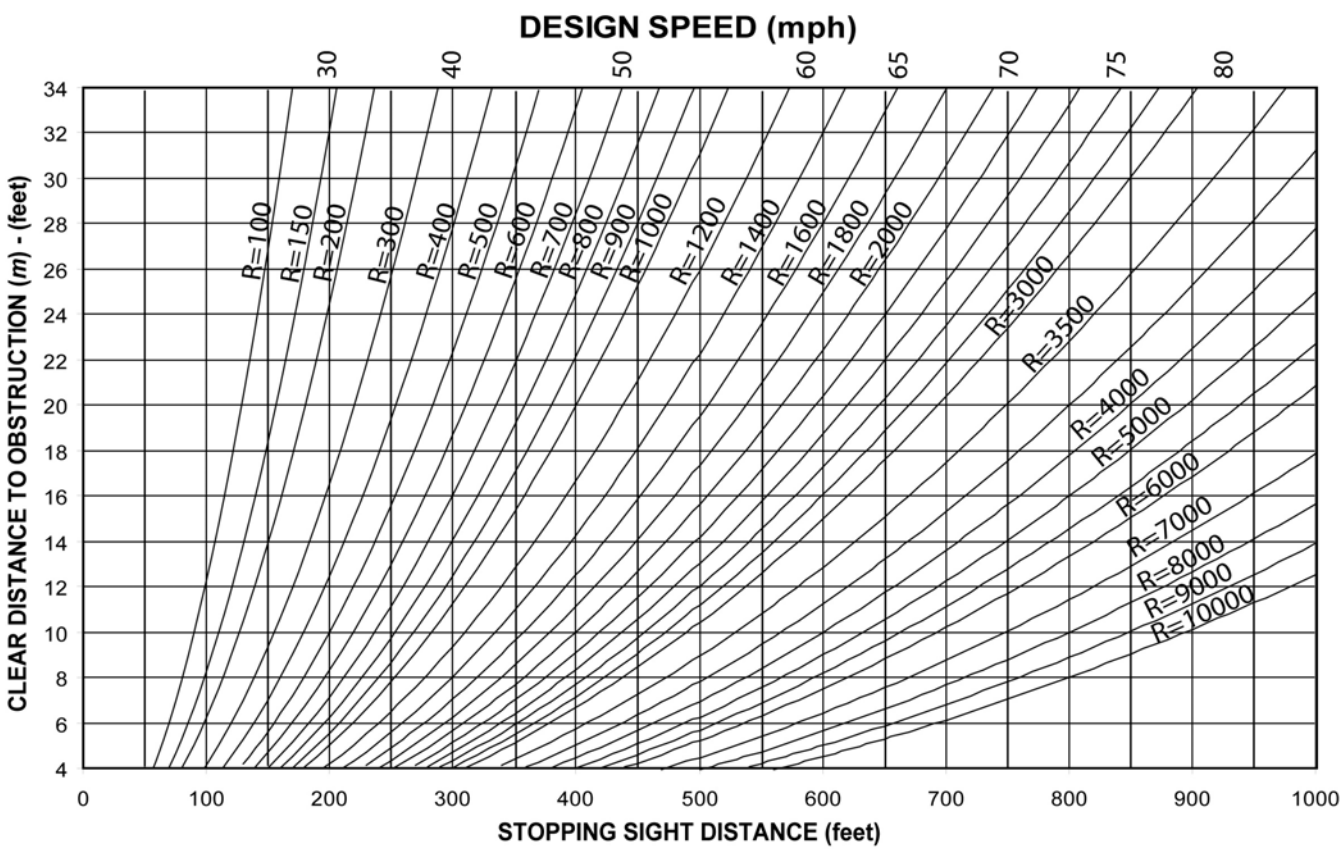
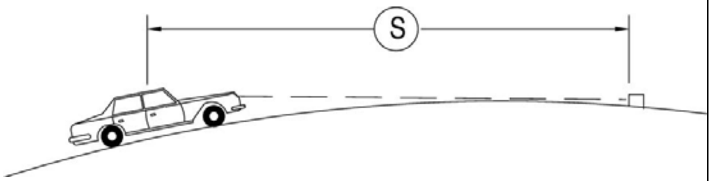


Figure 201.7
Decision Sight Distance on Crest Vertical Curves



Drivers eye height is 3½ feet.
Object height is ½-foot.

L = Curve Length (feet)

A = Algebraic Grade Difference (%)

S = Sight Distance (feet)

V = Design Speed for "S" in mph

K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Notes:

- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve.

When S > L	**When S < L**
L = 2S – 1329/A	**L = AS^2 /1329**

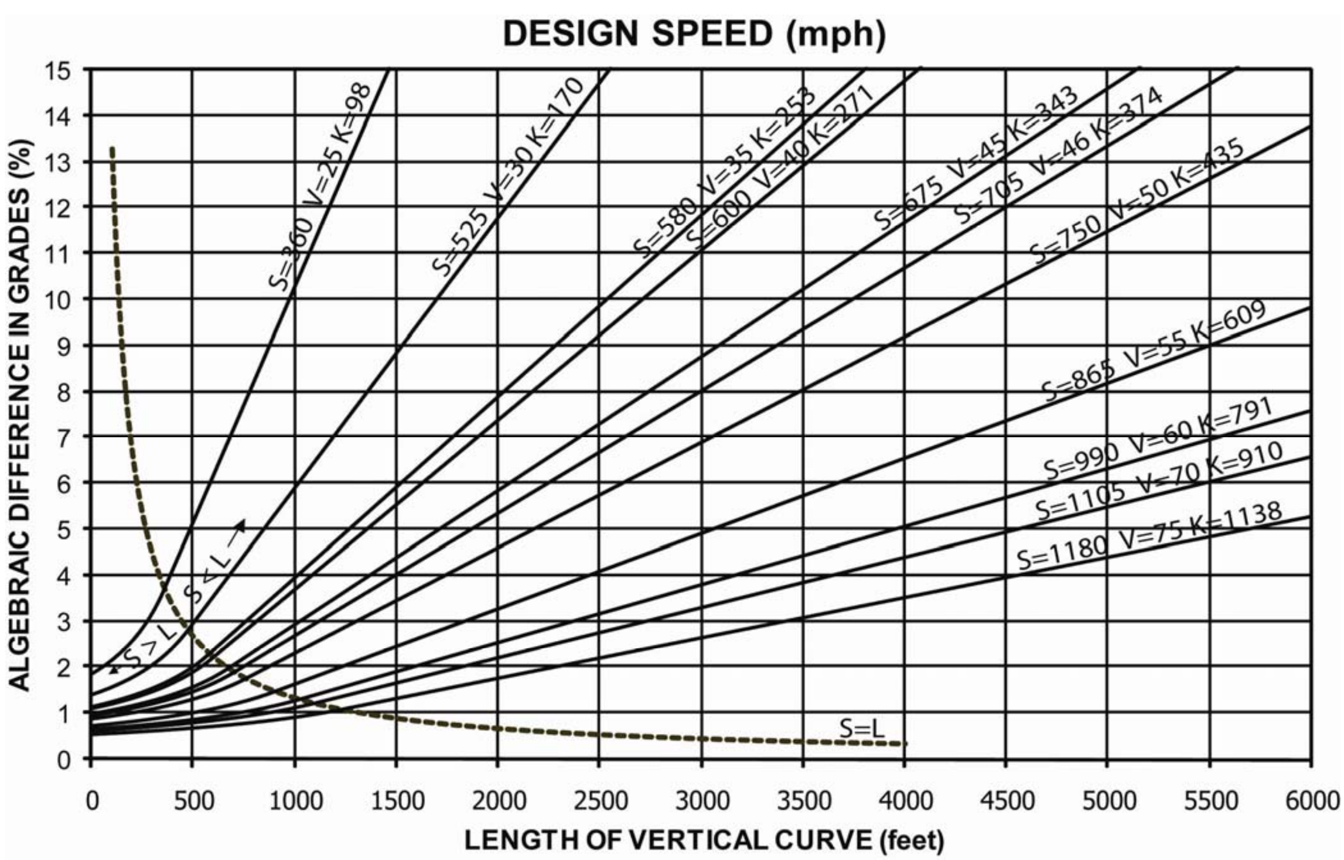
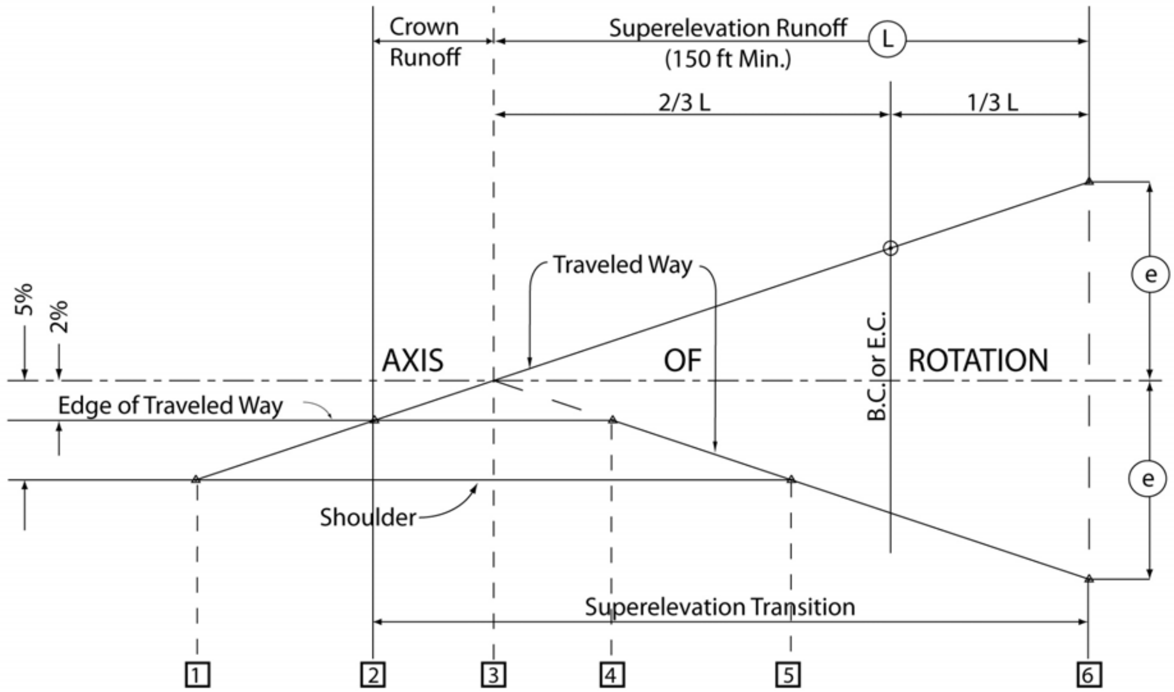


Figure 202.5A
Superelevation Transition

Formulas		Explanation of Terms
2-Lane Roads	L = 2500 e	(L) = Length of Superelevation Runoff - ft
Multilane Roads & Branch Connections	L = 150 De	(e) = Superelevation rate - ft/ft
Ramps Multilane Single Lane	 L = 2500 e if possible L = 2000 e	(D) = Distance from axis of rotation to outside edge of lanes - ft
MINIMUM L = 150 FT		MAXIMUM L = 510 FT

Adjust computed length to nearest 10 ft. length divisible by 3



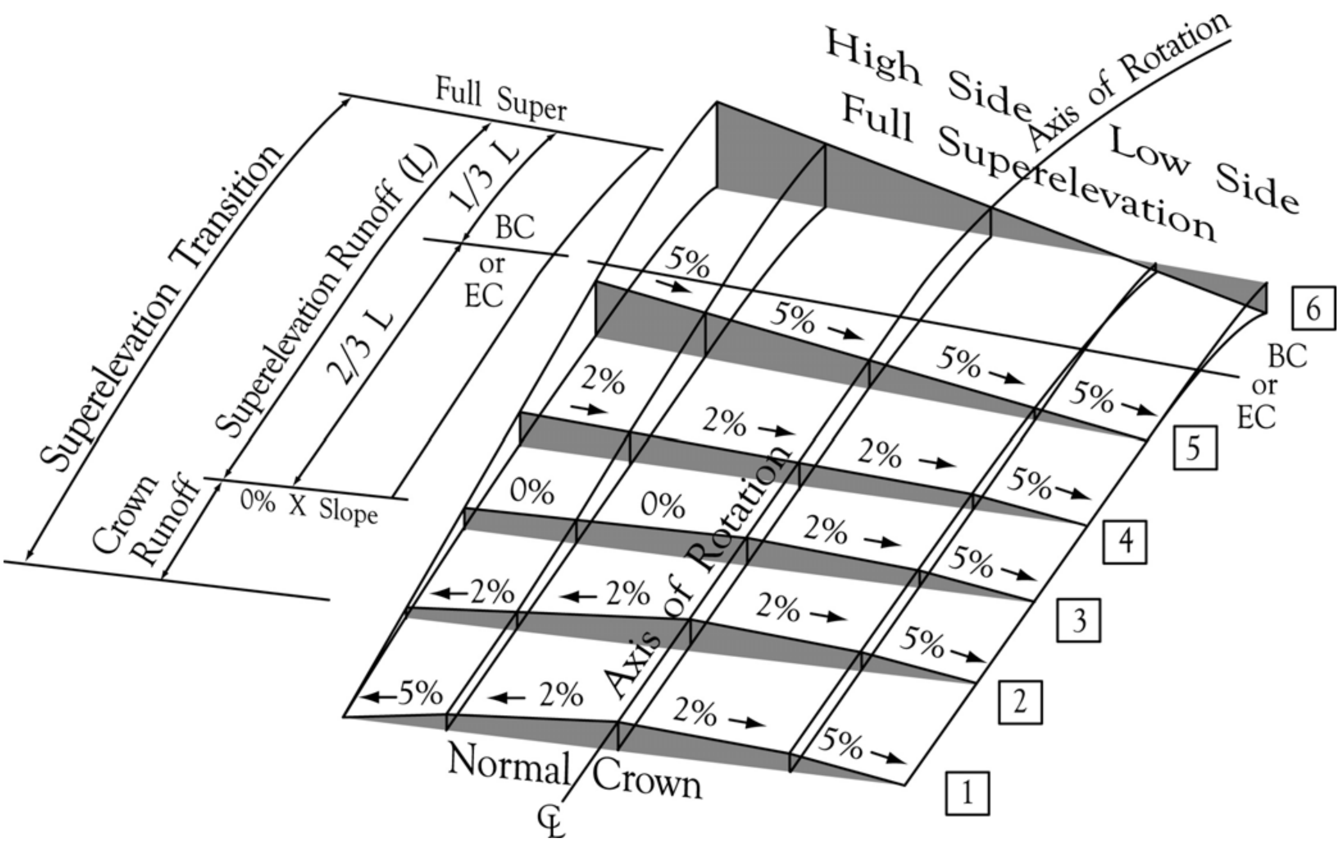
Superelevation Runoff Lengths

Superelevation Rate "e" ft/ft	Length, L (feet): 2-Lane Highways & Multilane Ramps	Single Lane Ramps	Multilane Highways and Branch Connections with Various "D" Widths: 24 ft	36 ft	48 ft	51 ft	60 ft	63 ft	75 ft
0.02	150	150	150	150	150	150	180	180	240
0.03	150	150	150	180	210	240	270	270	330
0.04	150	150	150	210	300	300	360	390	450
0.05	150	150	180	270	360	390	450	480	510
0.06	150	150	210	330	450	450	510	510	
0.07	180	150	270	390	510	510			
0.08	210	150	300	450					
0.09	240	180	330	480					
0.10	240	210	360	510					
0.11	270	210	390						
0.12	300	240	420						

For widths of "D" not included in table, use formula above.

Figure 202.5B
Superelevation Transition Terms & Definitions

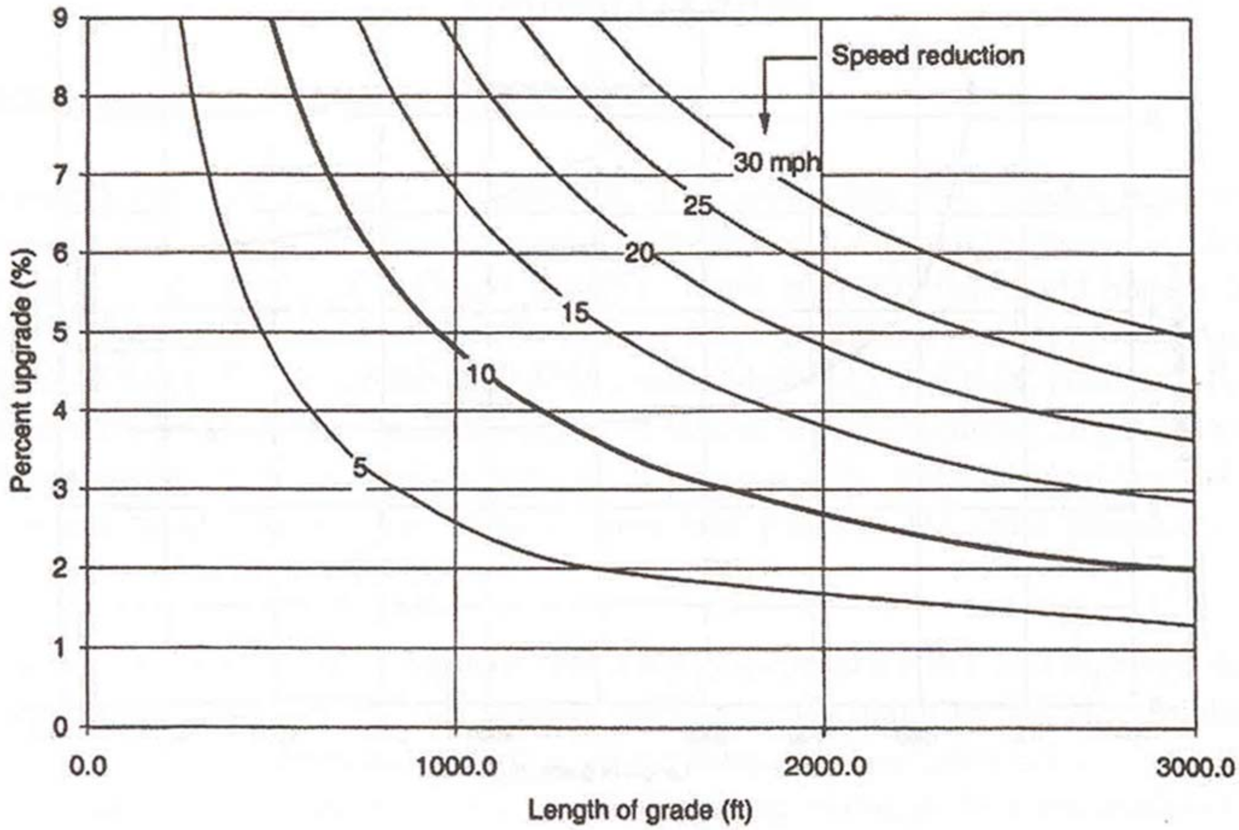
Term	Definition
Crown Runoff 2 ↔ 3	The distance from the station where the high side of the superelevating section surfaces are at a cross slope of 2% to where the high side of the section surfaces reaches a cross slope of 0%.
Superelevation Runoff(L) 3 ↔ 6	The distance from the station where the high side of the superelevating section surfaces are at a cross slope of 0% to the station where the entire cross section is at full superelevation.
Superelevation Transition 2 ↔ 6	The distance from the station where the high side of the superelevating sections are crowned at a cross slope of 2% to the station where the entire cross section is at full superelevation. The Crown Runoff Length plus the Superelevation Runoff Length (L) equals the Superelevation Transition Length.
% On tangent	The percentage of the superelevation runoff length (L) that is outside of the curve (2/3L). See Index 202.5(2).
% On curve	The percentage of the superelevation runoff length (L) that is within the curve (1/3L). See Index 202.5(2). The % On Tangent and % On curve values must total 100%.



Elements of a Superelevation Transition (Right Curve)

Figure 204.5

Critical Lengths of Grade for Design



ASSUMED TYPICAL HEAVY TRUCK OF 200 lb/hp

204.6 Coordination of Horizontal and Vertical Alignment

A proper balance between curvature and grades should be sought. When possible, vertical curves should be superimposed on horizontal curves. This reduces the number of sight restrictions on the project, makes changes in profile less apparent, particularly in rolling country, and results in a pleasing appearance. Where the change in horizontal alignment at a grade summit is moderate, a pleasing appearance may be attained by making the vertical curve overlap the horizontal curve.

When horizontal and vertical curves are superimposed, the combination of superelevation and profile grades may cause distortion in the outer pavement edges which could create drainage concerns or confuse drivers at night. In such situations edge of pavement profiles should be plotted and smooth curves introduced to eliminate any irregularities or distortion.

On highways in mountainous or rolling terrain where horizontal and vertical curves are superimposed at a grade summit or sag, the design speed of the horizontal curve should be at least equal to that of the crest or sag, and not more than 10 miles per hour less than the measured or estimated running (85th percentile) speed of vehicles on the approach roadway.

On long open curves, a uniform grade line should be used because a rolling profile makes for a poor appearance.

Horizontal and vertical curvature at intersections should be as flat as physical conditions permit.

See "Combination of Horizontal and Vertical Alignment" in Chapter III of AASHTO, A Policy on Geometric Design of Highways and Streets, for further guidance on a alignment consistency.

204.7 Separate Grade Lines

Separate or independent grade lines are appropriate in some cases for freeways and expressways.

They are not normally considered appropriate where medians are less than 65 feet wide (see Index 305.6). Exceptions to this may be minor differences between opposing grade lines in special situations.

In addition, for either interim or ultimate expressways, any appreciable grade differential between roadbeds should be avoided in the vicinity of at-grade intersections. For traffic entering from the crossroad, confusion and wrong-way movements could result if the pavement of the far roadway is obscured because of excessive grade differential.

204.8 Grade Line of Structures

(1) *Structure Depth.* The depth to span ratio for each structure is dependent on many factors. Some of these are: span, type of construction, aesthetics, cost, falsework limitations, and vertical clearance limitations. For purposes of preliminary planning and design, the depth to span ratios listed below may be used in setting grade lines at grade separations.

(a) Railroad Underpass Structures.

- Single track, through girder type structures: use 5-foot depth from top of rail to structure soffit (bottom of girder).
- Deck-type structures: for simple spans use d/s (depth to span ratio)= 0.08; for continuous multiple span structures use d/s= 0.07. These ratios do not include the additional 2 feet required above the deck for ballast and rail height.

(b) Highway Structures.

- Structures with single spans of 100 feet or less, use d/s= 0.06.
- Structures with single spans between 100 feet and 180 feet use d/s= 0.045.
- Continuous structures with multiple spans of 100 feet or less, use d/s= 0.055.
- Continuous structures with multiple spans of more than 100 feet, use d/s= 0.04.

Table 204.8
Falsework Span and Depth Requirements

Facility to be Spanned	Minimum Normal Width of Traffic Opening	Opening Width Provides for	Resulting Falsework Normal Span[1]	Depth of Superstructure[4]: Up to 6 feet	Up to 8 feet	Up to 10 feet	Up to 12 feet
				Minimum Falsework Depth			
Freeway	25'	1 Lane + 8' & 5' Shoulders	33'	1'-10½"	2'-1'	2'-1'	2'-8½"
	37'	2 Lanes + 8' & 5' Shoulders	45'	2'-9"	2'-11½"	3'-0"	3'-3"
	49'	3 Lanes + 8' & 5' Shoulders	57'	3'-3"	3'-3½"	3'-3½"	3'-3½"
	61'	4 Lanes + 8' & 5' Shoulders	69'	3'-4"	3'-5"	3'-7"	3'-7½"
Nonfreeway	20'	1 Lane + 2-4' Shoulders	28'	1'-9"	1'-10"	1'-10"	1'-10½"
	32'	2 Lanes + 2-4' Shoulders	40'	2'-0"	2'-8½"	2'-9"	3'-0"
	40'	2 Lanes + 2-8' Shoulders	48'	3'-0"	3'-0"	3'-2½"	3'-3"
	52'	3 Lanes + 2-8' Shoulders	60'	3'-3"	3'-3½"	3'-3½"	3'-4"
	64'	4 Lanes + 2-8' Shoulders	72'	3'-5"	3'-7½"	3'-7½"	3'-8"
Special	20'	1 Lane + 2-4' Shoulders	20'[3]	1'-9"	1'-10"	1'-10"	1'-10½"
Roadways[2]	32'	2 Lanes + 2-4' Shoulders	32'[3]	2'-0"	2'-8½"	2'-9"	3'-0"

Notes:

(1) Includes 8' for 2 temporary K-rails and deflection space.
(2) Uses such as fire or utility access or quasi-public roads with very light traffic.
(3) No temporary K-rail provided.
(4) See Index 204.8 for preliminary depth to span ratios. For more detailed information, contact the Division of Engineering Services, Structure Design and refer to the Bridge Design Aids.

Geometric plans should be submitted to the DOS prior to preparation of the Project Report so that preliminary studies can be prepared. Preliminary bridge type selection should be a joint effort between the DOS and the District.

(2) *Steel or Precast Concrete Structures.* Steel and precast concrete girders in lieu of cast-in-place concrete eliminate falsework, and may permit lower grade lines and reduced approach fill heights. Potential cost savings from elimination of falsework, lowered grade lines, and the ability to accomodate settlement beneath the abutments should be considered in structure type selection along with unit price, aesthetics, uniformity, and any other relevant factors. Note that grade lines at grade separations frequently need to be adjusted after final structure depths are determined (see

Index 309.2(3)). Details of traffic handling and stage construction should be provided when the bridge site plan is submitted to the DOS if the design or construction of the structure is affected (see Drafting and Plans Manual, Section 3-3.2).

(3) *Depressed Grade Line Under Structures.* Bridge and drainage design will frequently be simplified if the low point in the grade line is set a sufficient distance from the intersection of the centerlines of the structure and the highway so that drainage structures clear the structure footings.

(4) *Grade Line on Bridge Decks.* Vertical curves on bridge decks should provide a minimum fall of 0.05-foot per station. This fall should not extend over a length greater than 100 feet. The flattest allowable tangent grade should be 0.3 percent.

(5) *Falsework.* In many cases, it is economically justified to have falsework over traffic during construction in order to have a support-free open area beneath the permanent structure. The elimination of permanent obstructions usually outweighs objections to the temporary inconvenience of falsework during construction.

Because the width of traffic openings through falsework can, and oftentimes does, significantly affect costs, special care should be given to determining opening widths. The following should be considered: staging and traffic handling requirements, the width of approach roadbed that will exist at the time the bridge is constructed, traffic volumes, desires of the local agencies, controls in the form of existing facilities, and the practical problems of falsework construction.

<u>The normal minimum width of traffic openings and required falsework spans for various lane and shoulder combinations should be as shown in Table 204.8.</u>

When temporary K-rail is used to protect the falsework, space must be provided for its deflection. The normal spans shown in Table 204.8 provide 2 feet for this deflection.

In special cases, where existing constraints make it impractical to comply with the minimum widths of traffic openings set forth in Table 204.8, a lesser width may be approved by the District Director with concurrence from the Headquarters Design Coordinator.

The minimum vertical falsework clearance over freeways and nonfreeways shall be 15 feet. The following items should be considered:

- Mix, volume, and speed of traffic.
- Effect of increased vertical clearance on the grade of adjacent sections.
- Closing local streets to all traffic or trucks only during construction.
- Detours.
- Carrying local traffic through construction on subgrade.
- Temporary or permanent lowering of the existing facility.
- Cost of higher clearance versus cost of traffic control.
- Desires of local agency.

Worker safety should be considered when determining vertical falsework clearance. Requests for approval of temporary vertical clearances less than 15 feet should discuss the impact on worker safety.

Temporary horizontal clearances less than shown in Table 204.8 or temporary vertical clearances less than 15 feet should be noted in the PS&E Transmittal Report.

To establish the grade of a structure to be constructed with a falsework opening, allowance must be made for the depth of the falsework. The minimum depths required for various widths of traffic opening are shown in Table 204.8.

Where vertical clearances, either temporary or permanent are critical, the District and the DOS should work in close conjunction during the early design stage when the preliminary grades, structure depths, and falsework depths can be adjusted without incurring major design changes.

Where the vertical falsework clearance is less than 15 feet, advance warning devices are to be specified or shown on the plans. Such devices may consist of flashing lights, overhead signs, over-height detectors, or a combination of these or other devices.

Warning signs on the cross road or in advance of the previous off-ramp may be required for overheight permit loads. Check with the Regional Permit Manager.

After establishing the opening requirements, a field review of the bridge site should be made by the District designer to ensure that existing facilities (drainage, other bridges, or roadways) will not conflict with the falsework.

The placement and removal of falsework requires special consideration. During these operations, traffic should either be stopped for short intervals or diverted away from the span where the placement or removal operations are being performed. The method of traffic handling during these operations is to be included in the Special Provisions.

Topic 205 - Road Connections and Driveways

205.1 Access Openings on Expressways

Access openings are used only on expressways. The term access opening applies to openings through the right of way line which serve abutting land ownerships whose remaining access rights have been acquired by the State.

(1) *Criteria for Location.* <u>Access openings should not be spaced closer than one-half mile to an adjacent public road intersection or to another private access opening that is wider than 30 feet.</u> When several access openings are closely spaced, a frontage road should be considered (see Index 104.3). <u>To discourage wrong-way movements, access openings should be located directly opposite, or at least 300 feet from a median opening.</u>

    Sight distance equivalent to that required for public road intersections shall be provided (see Index 405.1).

(2) *Width.* The normal access opening width should be 30 feet. A greater width may result in large savings in right of way costs in some instances, but should be considered with caution because of the possibility that public use might develop. Conversion of a private opening into a public road connection requires the consent of the CTC, which cannot be committed in advance (see the Project Development Procedures Manual).

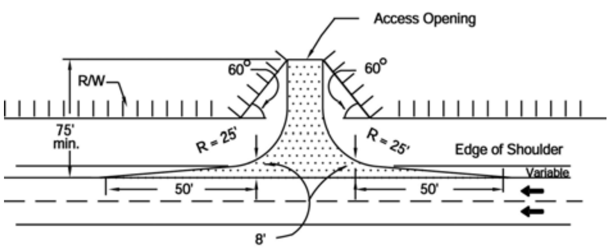
(3) *Recessed Access Openings.* Recessed access openings, as shown on Figure 205.1, are desirable at all points where private access is permitted and should be provided whenever they can be obtained without requiring alterations to existing adjacent improvements. When recessed openings are required, the opening should be located a minimum distance of 75 feet from the nearest edge of the traveled way.

(4) *Joint Openings.* A joint access opening serving two or more parcels of land is desirable whenever feasible. If the property

line is not normal to the right of way line, care should be taken in designing the joint opening so that both owners are adequately served.

(5) *Surfacing.* All points of private access should be surfaced with adequate width and depth of pavement to serve the anticipated traffic. The surfacing should extend from the edge of the traveled way to the right of way line.

Figure 205.1
Access Openings on Expressways



RECESSED OPENING

NOTES:

- By widening the expressway shoulder, deceleration lanes may be provided where justified.
- This detail, without the recess, may be used on conventional highways.

205.2 Private Road Connections

The minimum private road connection design is shown on Figure 205.1. Sight distance requirements for the minimum private road connection are shown on Figure 405.7 (see Index 405.1(2)(c)).

205.3 Urban Driveways

These instructions apply to the design of driveways to serve property abutting on State highways in cities or where urban type development is encountered.

Details for driveway construction are shown on the Standard Plans. Corner sight distance requirements are not applied to urban driveways. See Index 405.1(2) for further information.

(1) *Correlation with Local Standards.* Where there is a local requirement regulating driveway construction, the higher standard will normally govern.

(2) *Driveway Width.* The width of driveways for both residential and commercial usage is measured at the throat, exclusive of any flares. ("W" as shown in Standard Plan A87A).

(3) *Residential Driveways.* The width of single residential driveways should be 12 feet minimum and 20 feet maximum. The width of a double residential driveway such as used for multiple dwellings should be 20 feet minimum and 30 feet maximum. The width selected should be based on an analysis of the anticipated volume, type and speed of traffic, location of buildings and garages, width of street, etc.

(4) *Commercial Driveways.* Commercial driveways should be limited to the following maximum widths:

(a) When the driveway is used for one-way traffic, the maximum width should be 25 feet. If the driveway serves a large parcel, where large volumes of vehicles or large vehicles are expected, the entrance maximum width should be 40 feet and the exit maximum width should be 35 feet.

(b) When the driveway is used for two-way traffic, the maximum width should be 35 feet. If the driveway serves a large parcel, where large volumes of vehicles or large vehicles are expected, then the maximum width should be 45 feet.

(c) When only one driveway serves a given property, in no case should the width of the driveway including the side slope distances exceed the property frontage.

(d) When more than one driveway is to serve a given property, the total width of all driveways should not exceed 70 percent of the frontage where such a frontage is 100 feet or less. Where the frontage is more than 100 feet, the total driveway width should not exceed 60 percent of the frontage. In either case, the width of the individual driveway should not exceed those given in the preceding paragraphs. Where more than one driveway is necessary to serve any one property, not less than 20 feet of full height curb should be provided between driveways. This distance between driveways also applies to projects where curbs and gutters are not to be placed.

(e) Certain urban commercial driveways may need to accommodate the maximum legal vehicle. The width will be determined by the use of truck turn templates.

(5) Surfacing. Where curbs, gutters, and sidewalks are to be placed, driveways should be constructed of portland cement concrete. Where only curbs and gutters are to be placed and pedestrian traffic or adjacent improvements do not warrant concrete driveway construction, the driveway may be paved with the same materials used for existing surfacing on the property to be served.

(6) Pedestrian and Disabled Persons Access. Where sidewalks traverse driveways, accessibility regulations require that a relatively level (2 percent max. cross fall) path, at least 4 feet wide, is provided. Provision of this feature, as indicated in the Standard Plans, may require the acquisition of a construction easement or additional right of way. Assessment of these needs must be performed early enough in the design to allow time for acquiring any necessary permits or right of way. Additionally, designers should consider the following:

- Where restricted parking zones have been established (either blue or white painted zones) adjacent to driveways, but no reasonably close ramp access to the sidewalk exists, consideration should be given to reducing the maximum slope of the driveway from 10 percent to 8.33 percent to provide sidewalk access to the disabled.
- In many cases providing the pathway along the back of the driveway will lower the elevation at the back of the sidewalk. Depending on grades behind the sidewalk the potential may exist for roadway generated runoff to enter private property. The need for features such as low berms within the construction easement, or installation of catch basins upstream of the driveway should be determined.

When pedestrian activity is neither present, nor expected to be present within the reasonable future, the designer may develop driveway details that eliminate the flatter portion along the back edge in lieu of using the Standard Plans for driveways. Refer to Topic 105 for additional information related to pedestrian facilities.

205.4 Driveways on Frontage Roads and in Rural Areas

On frontage roads and in rural areas where the maximum legal vehicle must be accommodated, standard truck-turn templates should be used to determine driveway widths where the curb or edge of traveled way is so close to the right of way line that a usable connection cannot be provided within the standard limits.

Where county or city regulations differ from the State's, it may be desirable to follow their regulations, particularly where jurisdiction of the frontage road will ultimately be in their hands.

Details for driveway construction are shown on the Standard Plans. For corner sight distance, see Index 405.1(2)(c).

205.5 Financial Responsibility

Reconstructing or relocating any access openings, private road connections, or driveways required by revisions to the State highway facility should be done at State expense by the State or its agents. Reconstruction or relocation requested by others should be paid for by the requesting party.

Topic 206 - Pavement Transitions

206.1 General Transition Standards

Pavement transition and detour standards should be consistent with the section having the features built to the highest design standards. The transition should be made on a tangent section whenever possible and should avoid locations with horizontal and vertical sight distance restrictions. Whenever feasible, the entire transition should be visible to the driver of a vehicle approaching the narrower section. The design should be such that intersections at grade within the transition area are avoided. For decision sight distance at lane drops, see Index 201.7.

206.2 Pavement Widenings

(1) *Through Lane Additions.* Where through lanes, climbing lanes, or passing lanes are added, the minimum recommended distance over which to transition traffic onto the additional width is 250 feet per lane. Figure 206.2 shows several examples of acceptable methods for adding a lane in each direction to a two-lane highway.

(2) *Turning, Ramp, and Speed Change Lanes.* Transitions for lane additions, either for left or right turns or to add a lane to a ramp, should typically occur over a length of 120 feet. Lengths shorter than 120 feet are acceptable where design speeds are below 45 miles per hour or for conditions as stated in Index 405.2(2)(c).

Where insufficient median width is available to provide for left turn lanes, through traffic will have to be shifted to the outside. See Figures 405.2A, B and C for acceptable methods of widening pavement to provide for median turn lanes.

(3) *Lane Widening.* An increase in lane width can occur at short radius curves which are widened for truck off-tracking, at ramp terminals with large truck turning volumes, or when new construction matches existing roadways with narrow lane widths. Extensive transition lengths are not necessary as the widening does not restrict the drivers expectations. Transition tapers for these types of situations should be at 10:1 (longitudinal to lateral).

(4) *Shoulder Widening.* Shoulder widening should normally be accomplished in a manner that provides a smooth transition, but can be accomplished without a taper if necessary.

206.3 Pavement Reductions

(1) *Through Lane Drops.* <u>When a lane is to be dropped, it should be done by tapering over a distance equal to WV, where W = Width of lane to be dropped and V = Design Speed</u>. In general, the transition should be on the right so that traffic merges to the left. Figure 206.2 provides several examples of acceptable lane drops at 4-lane to 2-lane transitions. The exception to using the WV criteria is for the lane drop/freeway merge movement on a branch connection which is accomplished using a 50:1 taper.

(2) *Ramp and Speed Change Lanes.* As shown in Figures 504.2A and 504.3L, the standard taper for a ramp merge into a through traffic lane is 50:1 (longitudinal to lateral). Where ramp lanes are dropped prior to the merge with the through facility, the recommended taper is 50:1 for design speeds over 45 miles per hour, and the taper distance should be equal to WV for speeds below 45 miles per hour.

The "Ramp Meter Design Guidelines" also provide information on recommended and minimum tapers for ramp lane merges. These guideline values are typically used in retrofit or restricted right-of-way situations, and are acceptable for the specific conditions stated in the guidelines.

Figure 405.9 shows the standard taper to be used for dropping an acceleration lane at a signalized intersection. This taper can also be used when transitioning median acceleration lanes.

Figures 405.2A, B and C show the recommended methods of transitioning pavement back into the median area on conventional highways after the elimination of left-turn lanes.

Figure 309.2
Department of Defense
Rural and Single Interstate Routes

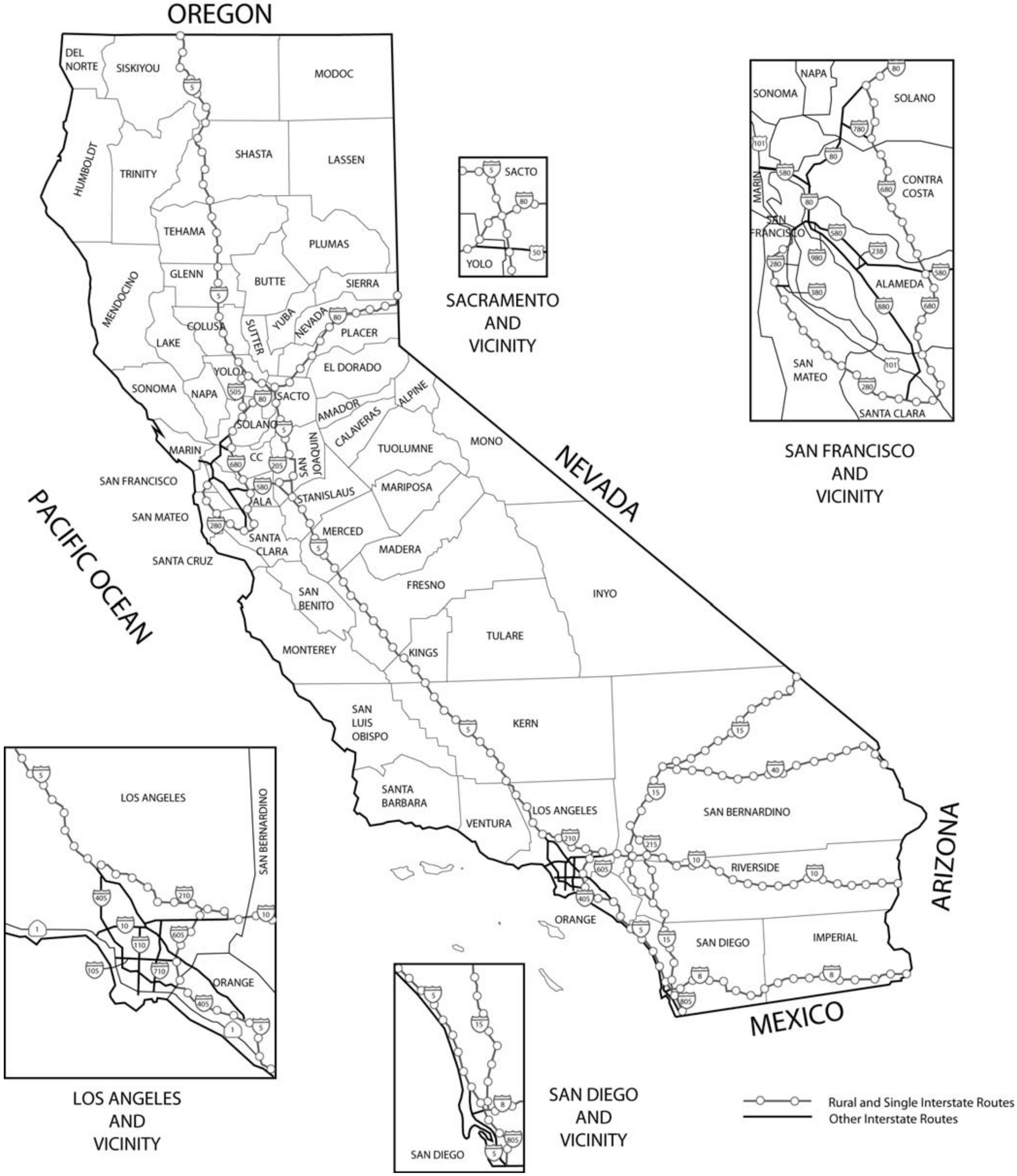


Table 309.2B
California Routes on the Rural and Single Interstate Routing System

ROUTE	FROM	TO
I-5	U. S. Border	I-805 just N. of U. S. Border
I-5	I-805 N. of San Diego	I-405 near El Toro
I-5	I-210 N. of Los Angeles	Oregon State Line
I-8	I-805 near San Diego	Arizona State Line
I-10	I-210 near Pomona	Arizona State Line
I-15	I-8 near San Diego	Nevada State Line
I-40	Junction at I-15 near Barstow	Arizona State Line
I-80	I-680 near Cordelia	Nevada State Line
I-205	Junction at I-580	Junction at I-5
I-210	I-5 N. of Los Angeles	I-10 near Pomona
I-215	I-15 near Temecula	I-15 near Devore
I-280	Junction at I-680 in San Jose	At or near south city limits of San Francisco to provide access to Hunter's Point
I-405	I-5 near El Toro	Palo Verde Avenue just N. of I-605
I-505	Junction at I-80	Junction at I-5
I-580	I-680 near Dublin	Junction at I-5
I-605	I-405 near Seal Beach	I-210
I-680	Junction at I-280 in San Jose	I-80 near Cordelia
I-805	I-5 just N. of U. S. Border	I-5 N. of San Diego

for single lane ramps or along the outside lane line for multilane ramps, with a central angle greater than 60 degrees, the single ramp lane, or the lane furthest to the right if the ramp is multilane, shall be widened in accordance with Table 504.3A in order to accommodate large truck wheel paths (see Topic 404). Consideration may be given to widening more than one lane on a multilane ramp with short radius curves if there is a likelihood of considerable bus or truck usage of that lane.

Table 504.3A
Ramp Widening for Trucks

Ramp Radius (ft)	Widening (ft)	Lane Width (ft)
<150	6	18
150 – 179	4	16
180 – 209	3	15
210 – 249	2	14
250 – 299	1	13
>300	0	12

(c) Shoulder Width--**Shoulder widths for ramps shall be as indicated in Table 302.1.** Typical ramp shoulder widths are 4 feet on the left and 8 feet on the right.

(d) Lane Drops--Typically, lane drops are to be accomplished over a distance equal to WV. Where ramps are metered, the recommended lane drop taper past the meter limit line is 50 to 1 (longitudinal to lateral). Where conditions preclude the use of a 50 to 1 taper, <u>the lane should be dropped using a taper of no less than 30 to 1</u>. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

<u>Lane drop tapers should not extend beyond the 6-foot point (the beginning of the weaving length) without the provision of an auxiliary lane.</u>

(e) Lane Additions -- Lane additions to ramps are usually accomplished by use of a 120-foot bay taper. See Table 405.2A for the geometrics of bay tapers.

(2) Ramp Metering

All geometric designs for ramp metering installations must be discussed with the Design Coordinator or Design Reviewer from the Division of Design. **Design features or elements which deviate from the mandatory standards require the approvals described in Index 82.2.** Before beginning any ramp meter design, the designer must contact District Traffic Operations for direction in the application of procedural requirements of the Division of Traffic Operations.

Geometric ramp design for new facilities should normally be based upon the projected peak-hour traffic volumes 20 years after completion of construction, except as stated in Index 103.2.

Geometric ramp design for operational improvement projects for ramp meters should be based on current peak-hour traffic volume (this is considered to be data that is less than two years old). If this data is not available it should be obtained before proceeding with design. Peak hour traffic data from the annual Traffic Volumes book is not adequate for this application.

The design advice and typical designs that follow should not be directly applied to ramp meter installation projects, especially retrofit designs, without giving consideration to "customizing" the geometric design features to meet site and traffic conditions (i.e., design highway volume, geometry, speeds, etc.). Every effort should be made by the designer to exceed the recommended minimum standards provided herein, where conditions are not restrictive.

(a) Metered Single-Lane Entrance Ramps

Geometrics for a single-lane ramp meter should be provided for volumes up to 900 vehicles per hour (vph) (see Figures 504.3A and 504.3B). <u>Where truck volumes (3-axle or more) are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3 percent</u>

(i.e., at least throughout the merge area), a minimum 500-foot length of auxiliary lane should be provided beyond the ramp convergence point. For additional guidance see AASHTO, A Policy on Geometric Design of Highways and Streets.

A multi-lane ramp segment may be provided to increase vehicle storage within the available ramp length (see 504.3(2)(d) Storage Length) and/or to create a preferential lane for HOVs, as required in Section 504.3(2)(h).

(b) Metered Multi-Lane Entrance Ramps

When entrance ramp volumes exceed 900 vph, and/or when an HOV lane is determined to be necessary, a two or three lane ramp segment should be provided. Figures 504.3C, 504.3D and 504.3E illustrate typical designs for metered two-lane ramps; and Figures 504.3F and 504.3G illustrate typical designs for metered three lane ramps. On two-lane loop ramps, normally only the right lane needs to be widened to accommodate design vehicle off-tracking. See 504.3(1)(b).

Three-lane metered ramps are typically needed to serve peak (i.e., commute) hour traffic along urban and suburban freeway corridors. The adverse effects of bus and truck traffic on the operation of these ramps (i.e., off-tracking, sight restriction, acceleration characteristics on upgrades, etc.) is minimized when the ramp alignment is tangential or consists of curve radii not less the 300 feet.

The recommended widths for metered ramps are shown in Table 504.3B.

On local street entrance ramps, the multi-lane segment should transition to a single lane width between the ramp meter limit line and the 6-foot separation point (from the mainline edge of traveled way). See Figures 504.3C, 504.3D, 504.3E, 504.3F, 504.3G, 504.3H and 504.3I.

Table 504.3B
Pavement Widths

Metered Ramp	Traveled Way	Inside Shoulder	Outside Shoulder
1-lane	12 ft	4 ft	8 ft
2-lane	24 ft	4 ft	8 ft
3-lane	36 ft	2 ft	2 ft

The lane drop transition should be accomplished with a taper of 50:1 (longitudinal to lateral) unless a lesser taper is warranted by site and/or project specific conditions which control the ramp geometry and/or anticipated maximum speed of ramp traffic. For example, "loop" entrance ramps would normally not allow traffic to attain speeds which would warrant a 50:1 (longitudinal to lateral) lane drop taper. Also, in retrofit situations, existing physical, environmental or right of way constraints may make it impractical to provide a 50:1 taper, especially if the maximum anticipated approach speed will be less than 50 miles per hour. Therefore, depending on approach geometrics and speed, the lane drop transition should be accomplished with a taper of between 30 and 50:1. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

Where truck volumes (3-axle or more) are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3 percent (i.e. at least throughout the merge area), a minimum 1,000 feet length of auxiliary lane should be provided beyond the ramp convergence point. AASHTO, A Policy on Geometric Design of Highways and Streets, provides additional guidance on acceleration lane length on grades.

When ramp volumes exceed 1,500 vph, a 1,000-foot minimum length of auxiliary lane should be provided beyond the ramp convergence point. If an auxiliary lane is related to the design of HOV preferential lane access.

Signing for an HOV preferential lane should be placed to clearly indicate which lane is designated for HOVs. Real-time signing at the ramp entrance, such as an overhead extinguishable message sign, may be necessary at some locations if pavement delineation and normal signing do not provide drivers with adequate lane usage information. To avoid trapping Single-Occupancy Vehicles (SOVs) in an HOV preferential lane, pavement delineation at the ramp entrance should lead drivers into the SOV lane.

(i) Modifications to Existing HOV Preferential Lanes

Changes in traffic conditions, proposals for interchange modifications, recurrent operational problems affecting the local facility, or the need to further improve mainline operations through more restrictive metering all provide an opportunity to reevaluate the need for an HOV preferential lane. HOV preferential lanes should remain in place or be added to the scope of projects generated in response to any of the above scenarios. Alternate solutions should be investigated before removal is considered. For examples: better control over ramp traffic can be attained by retrofitting ramps to meter HOV traffic which bypasses the ramp meter (District 3, 7, and 12). Underutilization of an existing lane plus the need for additional right of way for storage, the availability of an alternate HOV entrance ramp within 1½ mile, or the availability of a direct HOV access (drop) ramp will typically provide adequate justification for the removal of a preferential lane at specific locations.

The Deputy District Director of Operations, in consultation with the HQ Traffic Liaison, is responsible for approving decisions to remove HOV preferential lanes. Written documentation should be provided in the appropriate project document(s).

(j) Enforcement Areas and Maintenance Pullouts

Division of Traffic Operations policy requires an enforcement area be provided on all two-lane and three-lane on-ramps with HOV lanes. Deviation from this policy requires concurrence from the HQ Traffic Liaison, which must be reflected in the Project Initiation Document.

On single-lane ramps, a paved enforcement area is not necessary but the area should be graded to facilitate future ramp widening (see Figure 504.3A). Enforcement areas are used by the California Highway Patrol (CHP) to enforce vehicle occupancy requirements. At locations where the HOV lane is metered, the enforcement area should begin as close to the limit line as practical. Where unmetered, it should begin approximately 170 feet downstream of the limit line. On three-lane ramps, the enforcement area should be downstream of the mast arm standard, approximately 70 feet from the limit line. The length of the enforcement area and its distance downstream of the limit line may be adjusted to fit conditions at the ramp with CHP approval.

The District Traffic Operations Branch responsible for ramp metering shall coordinate enforcement issues with the California Highway Patrol. The CHP Area Commander shall be contacted during the Project Report stage, prior to design, to discuss any variations needed to the enforcement area designs shown in this manual. Variations shall be discussed with the HQ Traffic Liaison and the Design Coordinator and/or Design Reviewer.

A paved pullout area near the controller cabinet should be provided for safe and convenient access for Maintenance and Operations personnel. If a pullout cannot be provided, a paved or "all weather" walkway should be provided to the controller cabinet, see Index 107.2. See

Topic 309, Clearances, for placement guidance of fixed objects such as controller cabinets.

(3) *Location and Design of Ramp Intersections on the Crossroads.*

Factors which influence the location of ramp intersections on the crossroads include sight distance, construction and right of way costs, circuitous of travel for left-turn movements, crossroads gradient at ramp intersections, storage requirements for left-turn movements off the crossroads, and the proximity of other local road intersections.

Ramp terminals should connect where the grade of the overcrossing is 4 percent or less to avoid potential overturning of trucks.

For left-turn maneuvers from an off-ramp at an unsignalized intersection, the length of crossroads open to view should be greater than the product of the prevailing speed of vehicles on the crossroads, and the time required for a stopped vehicle on the ramp to execute a left-turn maneuver. This time is estimated to be 7½ seconds.

Where a separate right-turn lane is provided at ramp terminals, the turn lane should not continue as a "free" right unless pedestrian volumes are low, the right-turn lane continues as a separate full width lane for at least 200 feet prior to merging and access control is maintained for at least 200 feet past the ramp intersection. Provision of the "free" right should also be precluded if left-turn movements of any kind are allowed within 400 feet of the ramp intersection.

Horizontal sight restrictions may be caused by bridge railings, bridge piers, or slopes. Sight distance is measured between the center of the outside lane approaching the ramp and the eye of the driver of the ramp vehicle assumed 10 feet back from the edge of shoulder at the crossroads. Figure 504.3J illustrates the determination of ramp setback from an overcrossing structure on the basis of sight distance controlled by the bridge rail. The same relationship exists for sight distance controlled by bridge piers or slopes.

Where ramp set back for the 7½ second criterion is unobtainable, sight distance should be provided by flaring the end of the overcrossing structures or setting back the piers or end slopes of an undercrossing structure.

If signals are warranted within 5 years of construction, consideration may be given to installing signals initially in lieu of providing horizontal sight distance which meets the 7½ second criterion. See Part 4 of the California MUTCD, 4B.107(CA). However, this is not desirable and corner sight distance commensurate with design speed should be provided where obtainable (see AASHTO, A Policy on Geometric Design of Highways and Streets).

For additional information on sight distance requirements at signalized intersections, see Index 405.1.

For new construction or major reconstruction of interchanges, the minimum distance (curb return to curb return) between ramp intersections and local road intersections shall be 400 feet. The preferred minimum distance should be 500 feet. This does not apply to Resurfacing, Restoration and Rehabilitation (RRR), ramp widening, restriping or other projects which do not reconfigure the interchange. This standard does apply to projects proposing to realign a local street.

Where intersections are closely spaced, traffic operations are often inhibited by short weave and storage lengths, and signal phasing. In addition it is difficult to provide proper signing and delineation. Whenever it becomes necessary to locate a ramp terminal close to an intersection, the District Traffic Branch should be consulted regarding the requirement for signing, delineation and signal phasing.

(4) *Superelevation for Ramps.* The factors controlling superelevation rates discussed in Topic 202 apply also to ramps. As indicated in Table 202.2 use the 12 percent e_{max} rate except where snow and ice conditions prevail. In restrictive cases where the length of curve is too short to develop standard superelevation, the highest obtainable rate should be used (see

CHAPTERS 800 - 890
HIGHWAY DRAINAGE DESIGN

CHAPTER 800 - GENERAL ASPECTS

Topic 801 - General

Index 801.1 - Introduction

This section is not a textbook, and is not a substitute for fundamental engineering knowledge or experience.

The fields of hydrology and the hydraulics of highway drainage are rapidly evolving and it is the responsibility of the engineer to keep abreast of current design practices. As new practices or procedures are adopted by the Department, this section will be updated.

Instructions for the design of highway drainage features provided are for information and guidance of Department employees. Drainage policies, procedures and standards given are subject to amendment as conditions warrant and are neither intended as, nor do they establish, legal standards. Special situations may call for variations from these requirements, subject to approval of the Division of Design or approval by others as may be specifically referenced.

801.2 Drainage Design Philosophy

Highway drainage design is much more than the mere application of the technical principles of hydrology and hydraulics. Good drainage design is a matter of properly balancing technical principles and data with the environment giving due consideration to other factors such as safety and economics. Such design can only be accomplished through the liberal use of sound engineering judgment. Drainage features to remove runoff from the roadway and to convey surface and stream waters originating upstream of the highway to the downstream side should be designed to accomplish these functions without causing objectionable backwater, excessive velocities, erosion or unduly affecting traffic safety. A goal in highway drainage design should be to perpetuate natural drainage, insofar as practical.

801.3 Drainage Standards

Drainage design criteria should be selected that are commensurate with the relative importance of the highway, associated risks, and possible damage to adjacent property. The objective of drainage design should be to provide optimum facilities considering function versus cost rather than to just meet minimum standards.

Engineers and other professional disciplines using this guide must recognize that hydrologic analysis, as practiced by the highway engineer, has not advanced to a level of precise mathematical expression. All hydrologic analysis methods, whether deterministic or statistical, are based on the information available. A common challenge faced by the highway design engineer is that there may be insufficient flow data or no data at all at the site for which a stream crossing is to be designed. By applying analytical principles and methods it is possible to obtain peak discharge estimates which are functionally acceptable for the design of highway drainage structures and other features.

The design of highway drainage structures and other features must consider the probability of flooding and provide protection which is commensurate with the importance of the highway, the potential for property damage, and traffic safety. Traditionally, the level of assurance for such protection has been specified in terms of the peak rate of flow during passage of a flood or storm of the severity associated with the frequency of occurrence, i.e. a 10-year storm, the 50-year flood, etc. State-of-the-art methods and procedures associated with the necessary hydrologic analysis required to determine the severity and probability of occurrence of possible rare storms and flood events are inherently ambiguous. Therefore, the suggested drainage design criteria relating to frequency of occurrence references in this manual are provided for guidance only and are not intended to establish either legal or design standards which must be strictly adhered to. Rather, they are intended as a starting point of reference for designing the most cost effective drainage structures and facilities considering the importance of the highway, safety, legal obligations, ease of maintenance, and aesthetics.

801.4 Objectives of Drainage Design

Drainage design seeks to prevent the retention of water by the highway and provide for removal of water from the roadway through a detailed analysis considering all pertinent factors.

Specific steps to be taken generally include:

(a) Estimating the amount and frequency of storm runoff.

(b) Determining the natural points of concentration and discharge, the limiting elevations of entrance head, and other hydraulic controls.

(c) Estimating the amount and composition of bedload and its abrasive and bulking effects.

(d) Determining the necessity for protection from floating trash and from debris moving under water.

(e) Determining the requirements for energy dissipation and bank protections.

(f) Determining the necessity of providing for the passage of fish and recognizing other ecological conditions and constraints. Water quality and pollution control are discussed under Index 110.2. Aspects of wetlands protection are covered under Index 110.4.

(g) Analyzing the deleterious effects of corrosive soils and waters on structures.

(h) Comparing and coordinating proposed design with existing drainage structures and systems handling the same flows.

(i) Coordinating, with local agencies, proposed designs for facilities on roads to be relinquished.

(j) Providing access for maintenance operations.

(k) Providing for removal of detrimental amounts of water on traveled ways (see Topics 831 and 833).

(l) Providing for removal of detrimental amounts of subsurface water.

(m) Designing the most efficient drainage facilities consistent with the factors listed above, economic considerations, the importance of the highway, ease and economy of maintenance, engineering judgement, and aesthetics.

(n) Checking the structural adequacy of designs by referral to Structures Design or by use of data furnished by Structures Design.

(o) Preventing water from crossing slopes in concentrated flows.

801.5 Economics of Design

An economic analysis of alternate drainage designs, where a choice is available, should always be made. Non-engineering constraints may severely limit the design alternatives available to the drainage design engineer for a specific project or location. Generally, however, the design engineer has a wide range of materials and products to choose from in selecting the most economical design from available alternatives for highway drainage structures and other features.

The following factors should be considered in the selection of alternative designs and economic comparisons:

(a) Initial cost of construction and right of way.

(b) Evaluation of flood related risks to the highway and to adjacent properties including potential liabilities for damage.

(c) Cost of detours and traffic handling.

(d) Service life of the highway and of the drainage structure.

(e) Cost of providing traffic safety features.

(f) Aesthetics.

(g) Costs to traveling public for delays or extra travel distance due to road closures.

(h) Initial cost versus long term maintenance costs for cleanout, repair, traffic control and other pertinent maintenance charges that may be incurred during the life of the facility.

(i) Safety of required maintenance activities, ability to provide maintenance mechanically and to reduce worker exposure.

(j) Inlet and outlet treatment.

(k) Potential for causing erosion and effective water pollution control.

801.6 Use of Drainage References

No attempt has been made herein to detail basic hydrologic and hydraulic engineering techniques.

Various sources of information, including FHWA Hydraulic Engineering Circulars (HEC's); Title 23, Code of Federal Regulations (CFR), Part 650, Subpart A; AASHTO Guidelines; Federal-Aid Policy Guide and numerous hydrology and hydraulics reports and texts have been used to compile this highway drainage guide. Frequent references are made to these publications. Where there is a conflict in information or procedure, engineers must look at all pertinent parameters and use their best judgment, to determine which approach is the most consistent with the objectives of Caltrans drainage design principles and which most closely relates to the specific design problem or project.

Topic 802 - Drainage Design Responsibilities

802.1 Functional Organization

(1) *Division of Design.* The Office of State Highway Drainage Design in Division of Design performs the following functions under the direction of the Headquarters Hydraulics Engineer:

(a) Provide design information, guidance and standards to the Districts for the design of surface and subsurface drainage.

(b) Keep informed on the latest data from research, experimental installations, other public agencies, and industry that might lead to improvement in drainage design practices.

(c) Promote statewide uniformity of design procedures, and the exchange of information between Districts.

(d) Coordinate drainage design practices with other Caltrans Offices.

(e) Review special drainage problems and unusual drainage designs on the basis of statewide experience.

(f) Act in an advisory capacity to the Districts when requested.

(2) *Division of Engineering Services (DES).* The DES is responsible for:

(a) The hydraulic design of bridges, bridge deck drains, and special culverts.

(b) The structural adequacy of all drainage facilities.

(c) The adequacy of pumping plant characteristics and temporary storage. Refer to Topic 839 for further discussion on pumping stations.

(d) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 and submittal of preliminary hydraulic data as outlined under Topic 805.

(e) Geotechnical (soil mechanics and foundation engineering) considerations.

(3) *Legal Division.* The Legal Division provides legal advice and guidance to other Caltrans Offices concerning the responsibilities of the Department and owners of property along State highways with regard to surface water drainage.

(4) *Districts.* The District Director is responsible for:

(a) The hydrology for all drainage features except bridges.

(b) The hydraulic adequacy of all drainage features, except bridges and any special culverts and appurtenances designed by the Division of Engineering Services.

(c) Consulting with the Division of Engineering Services when it is proposed that an existing bridge be replaced with a culvert.

(d) Bank and shore protection designs, including erosion protection measures at ends of bridges and other structures designed by the Division of Engineering Srevices.

(e) Assigning one or more engineers in responsible charge of hydrologic study activities and the hydraulic design of drainage features.

(f) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 for storm drain systems.

(g) Providing additional staff as necessary with the training and background required to perform the following:

- Accomplish the objectives of drainage design as outlined under Index 801.4
- Prepare drainage plans or review plans prepared by others.
- Study drainage problems involving cooperative agreements and make recommendations to the decision makers.
- Accumulate and analyze hydrologic and hydraulic data reflecting the local conditions throughout the District for use in design.
- Review drainage changes proposed during construction.
- Make investigations and recommendations on drainage problems arising from the maintenance of existing State highways.
- Coordinate drainage design activities with other District Offices and Branches.
- Coordinate drainage designs with flood control districts and other agencies concerned with drainage by representing the District at meetings and maintaining an active liaison with these agencies at all times.
- Furnish data as required on special problems, bridges, large culverts, culverts under high fills and pumping plants that are to be designed by the Division of Engineering Services.
- Make field inspections of proposed culvert sites, existing drainage structures during storms, and storm damage locations.
- Document condition and file data that might forestall or defend future lawsuits.
- Review permits for drainage facilities to be constructed by other agencies or private parties within the highway right of way.
- Investigate and prepare responses to complaints relative to drainage conditions on or adjacent to the right of way.

Assignment of the duties described above will vary between districts. Due to the increasing complexity of hydraulic and hydrologic issues it is imperative that the more complex analyses be performed by experienced hydraulic designers. To provide guidance on those issues where district hydraulic units should become involved, the following list is provided.

- Storm drain design and calculations.
- Drainage basins exceeding 320 acres.
- Hydrograph development or routing.
- Open channel modification or realignment.
- Retention or detention basins.
- Backwater analysis.
- High potential for flood damage litigation.
- Scour analysis or sediment transport (typically forwarded to DOS).

- Culvert designs greater than 36 inches in diameter.
- Encroachments on FEMA designated floodplains.
- Modifications to inlet or outlet capacities on existing culverts or drainage inlets (e.g., placement of safety end grates, conversion of side opening inlets to grated inlets, etc.).
- Unique hydraulic design features (e.g., energy dissipator design, pumping stations, siphons, etc.).

This list is not all inclusive, and many additional functions are likely to be performed by hydraulic units. Although various constraints may preclude the hydraulic unit from actively performing the design or analysis of these items, a thorough review by that unit should be performed, at a minimum.

(5) *Materials Engineering and Testing Services.* METS provides advice and guidance to other Caltrans Offices and Branches concerning service life, physical properties, and structural adequacy of materials used in drainage design.

802.2 Culvert Committee

The Caltrans Culvert Committee is composed of nine members representing the Offices of State Highway Drainage Design, Structure Design, Office Engineer, and Materials Engineering and Testing Services, along with the Division of Construction and the Division of Maintenance. The Committee is chaired by the Headquarters Hydraulics Engineer in the Office of Highway Drainage Design. The Committee performs the following functions:

(a) Investigates new materials and new installation methods that may improve the economic service life of culverts and other drainage facilities.

(b) Coordinates drainage design practice with other headquarters departments.

(c) Follows current research and takes steps to implement successful findings.

(d) Acts as an advisory group to Districts and other Caltrans Offices when requested.

(e) Serves as Caltrans liaison with manufacturers, suppliers, contractors and industry associations.

The authority of the Committee is advisory only, and recommendations of the Committee are submitted to the Chief, Division of Design for approval and implementation through design guidelines and standards.

Requests for consideration of new materials, methods, or procedures should be directed to the Committee Chairman.

802.3 Bank and Shore Protection Committee

The Caltrans Bank and Shore Protection Committee is composed of representatives from DES Structures Maintenance and Investigation, Office of State Highway Drainage Design, METS, Division of Construction, and Division of Maintenance. It is chaired by the Office of Highway Drainage Design representative.

The Committee performs the following functions:

(a) Acts as a service and an advisory group available to Districts and Caltrans Offices and Branches upon written request for special investigations or study. Requests for special investigation of rock slope protection, channel or bridge protection, major channel changes, etc. should be directed to the Committee Chair.

(b) Provides conceptual input and acts as approval authority for supplements or modifications to bank and shore protection practice publications as warranted.

(c) Investigates and provides input toward the development of detailed design criteria for the various types of bank and shore protection.

(d) Observes performances of existing and/or experimental installations during or following severe exposures. The Districts or Caltrans Offices or Branches are requested to inform the Chair, Bank and Shore Protection Committee, or any available members of the Committee, of damage to installations by flood or high seas.

(e) Upon submission by the Department's New Products Coordinator, the Committee evaluates new products and processes related to bank and shore protection for possible approval.

Topic 803 - Drainage Design Policies

803.1 Basic Policy

In drainage design, the basic consideration is to protect the highway against damage from storm and subsurface waters, taking into account the effect of the proposed improvement on traffic and property. Unless the State would benefit thereby, or the cost is borne by others, no improvement in the drainage of areas outside the right of way is to be considered on Caltrans projects.

803.2 Cooperative Agreements

The extent of the department's financial participation in cooperative drainage improvement projects must be commensurate with the benefits to the Department and the traveling public.

(1) Local Agencies. Caltrans may participate with Local Agencies, Flood Control Districts or Drainage Assessment Districts on drainage improvement projects. Such projects must be covered by a formal agreement prepared and processed in accordance with instructions in the Caltrans Cooperative Agreement Manual.

(2) Federal and State Flood Control Projects. The cost of upgrading or modifying existing State highway facilities to accommodate Federal and/or State funded flood control projects is normally the responsibility of the agency funding the project. As necessary, Caltrans may enter into agreements containing provisions that the cost of betterments to existing highways, including drainage features, will be paid for by the Department. The Cooperative Agreement Manual contains procedures for preparing interagency agreements.

803.3 Up-Grading Existing Drainage Facilities

(1) Rehabilitation and Reconstruction Projects. The hydraulic adequacy, as well as the structural adequacy of existing drainage facilities should be evaluated early in the project development process on pavement rehabilitation and highway reconstruction projects.

Repair or replacement of structurally deficient drainage structures and up-grading of hydraulically inadequate drainage facilities should, whenever practicable, be included in the work of the proposed project. A thorough investigation of upstream and downstream conditions is often required to reveal what adverse effects there may be with increasing the capacity or velocity of existing cross drainage.

A cooperative agreement should be negotiated when the proposed work includes the upgrading of an existing storm drain system under the jurisdiction of a local or other public agency.

(2) Proposed Upstream Development. Unless developers of land in the drainage basin upstream of existing State highways incorporate positive stormwater management practices, such as detention or retention storage basins within their improvement areas, the peak flow from stormwater runoff is nearly always increased. As a practical matter, minor increases in peak flow are usually not objectionable. However, uncontrolled upstream development or diversions can significantly increase the peak flow run-off causing the passable capacity of the downstream drainage systems, including existing highway culverts, to be exceeded.

When reasonable solutions to potential drainage problems associated with such increased flows include the up-grading of drainage facilities within the State highway right-of-way, cooperative agreements with the responsible local agency should be negotiated. The local agency having permit authority has the responsibility for assessing liabilities and seeking commensurate funding for mitigation of run-off impacts from the developers. The local agency should not allow potentially harmful developments to proceed until all issues have been resolved. If it becomes apparent that the District, the local agency and the developer may not amiably reach agreement, the matter should be referred to Caltrans Legal Division before there is an impasse in the negotiations.

Caltrans financial participation in such drainage improvements must be based on the general rule stated in Index 803.2 Cooperative Agreements.

(3) *Hydraulically Inadequate Facilities.* Land use changes nearly always cause areas to become less pervious and drainage basins to yield greater volumes and increase peak stormwater run-off flows. Even development of a small parcel of land within a drainage basin causes some increase in stormwater run-off. Individually the increase may be negligible. Collectively these incrementally small increases over time may cause the design capacity of an existing culvert to be exceeded.

The up-grading of this category of hydraulically inadequate drainage facilities may be partially or fully financed by Caltrans. Only if the benefit cost (b/c) ratio is equal to or greater than one is up-grading viable for normal Caltrans project funding. When the benefits to the Department and the traveling public do not justify increasing the capacity, up-grading may still be accomplished cooperatively with the local agency in accordance with the general rule for participation under Index 803.2 Cooperative Agreements.

Topic 804 - Floodplain Encroachments

804.1 Purpose

The purpose of these instructions is to provide uniform procedures and guidelines for Caltrans multi-disciplinary evaluation of proposed highway encroachments on floodplains.

804.2 Authority

Title 23, CFR, Part 650, Subpart A, prescribes FHWA's "...policies and procedures for the location and hydraulic design of highway encroachments on floodplains, ...". The CFR's may be found on-line at: http://www.access.gpo.gov/nara/cfr/cfr-table-search.html

804.3 Applicability

The guidance provided herein establishes Caltrans procedures whenever a floodplain encroachment is anticipated. Adherence to these procedures will also ensure compliance with applicable Federal regulations which apply to any Federally approved highway construction, reconstruction, rehabilitation, repair, or improvement project which affects the (100-year) base floodplain. Work outside the limits of the base floodplain should be reviewed to see if it affects the (100-year) base floodplain. The only exception is repairs made during or immediately following a disaster. The premise is that all Federal-aid projects be evaluated and that diligent efforts be made to:

- Avoid significant floodplain encroachments where practicable.
- Minimize the impact of highway actions that adversely affect the base floodplain.
- Be compatible with the National Flood Insurance Program (NFIP) of the Federal Emergency Management Agency (FEMA).

804.4 Definitions

The following definitions of terms are made for the purpose of uniform application in the documentation and preparation of floodplain evaluation reports. Refer to Title 23, CFR, Part 650, Section 650.105 for a complete list of definitions.

(1) *Base Flood.* The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood).

(2) *Base Floodplain.* The area subject to flooding by the base flood. Every watercourse (river, creek, swale, etc.) is

subject to flooding and theoretically has a base floodplain.

(3) *Design Flood*. The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.

(4) *Encroachment.* An action within the limits of the base floodplain. Any construction activity (access road, building, fill slopes, bank or slope protection, etc.) within a base floodplain constitutes an encroachment.

(5) *Location Hydraulic Study.* A term from 23 CFR, Section 650.111 referring to the preliminary investigative study to be made of base floodplain encroachments by a proposed highway action. The extent of investigation and the discussion content in the required documentation of the "Location Hydraulic Study" is very site specific and need be no more than that which is commensurate with the risk(s) and impact(s) particular to the location under consideration. The information developed, documented (refer to Figure 804.7A) and retained in the project file is the suggested minimum necessary for compliance.

(6) *Natural and Beneficial Floodplain Values.* This shall include but is not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

(7) *Overtopping Flood.* The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

(8) *Regulatory Floodway.* The floodplain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program).

804.5 Procedures

Floodplain evaluations are essentially an extension of the environmental assessment process and instructions contained in the Environmental Handbook and the Project Development Procedures Manual are to be followed. Early in the planning of a project it is necessary to first determine:

(a) If a proposed route alternative will encroach on a base floodplain (refer to Index 804.4 (2)) or,

(b) Where proposed construction on existing highway alignment encroaches on a base floodplain.

A Location Hydraulic Study is used to determine (a) and (b) above. Refer to Index 804.4 (4) and 804.7 (2)(b) for further discussion.

Where National Flood Insurance Program (NFIP) Maps and study reports are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. Three types of NFIP maps are published which, if available, may be obtained from the District Hydraulics Branch: Flood Hazard Boundary Map (FHBM), Flood Boundary and Floodway Map (FBFM), and Flood Insurance Rate Map (FIRM).

If NFIP Maps are not available, the District Hydraulics Engineer should develop hydrologic data and hydraulic information to estimate the limits of the 100-year base floodplain to determine whether a highway location alternative will include an encroachment.

Projects which involve proposed construction within a regulatory floodplain or floodway need to be analyzed to determine whether it may be necessary to obtain a map revision. A map revision is required when construction in the floodplain increases the base flood elevation (BFE) more than 1 foot. Not all new construction projects require a map revision.

804.6 Responsibilities

The District Project Engineer is generally the responsible party for initiating and coordinating the overall multi-disciplinary team activities of evaluation and documentation of floodplain impacts. Discussion of specific hydraulic and environmental aspects are required by 23 CFR 650, Subpart A. Preparing the project floodplain evaluation report and the summary for the environmental document or project report is normally the responsibility of the Environmental Planning Branch. The District Hydraulics Engineer will, as necessary, develop the hydrological and hydraulic information and provide technical assistance for assessing impacts of floodplain encroachments.

804.7 Preliminary Evaluation of Risks and Impacts for Environmental Document Phase

Virtually all proposed highway improvements that are considered as floodplain encroachments will be designed to have:

(a) No significant risks associated with implementation and,

(b) Negligible environmental impacts on the base floodplain.

(1) Risks. There will always be some potential for property damage and flooding that may affect public safety, associated with highway drainage design. In a majority of cases, a field review with a NFIP or USGS map and the application of good engineering judgment are all that is needed to determine if such risks are significant or acceptable. The detail of study and documentation shall be commensurate with the risk(s) or floodplain impact(s) and, in all cases, should be held to the minimum necessary to address 23 CFR 650.111.

(2) Impacts. The assessment of potential impacts on the floodplain environment will include:

(a) Impacts on natural and beneficial floodplain values.

(b) Support of probable incompatible floodplain development.

Except for the more environmentally sensitive projects, a single visit to the project site by the District Project Engineer, Hydraulics Engineer, and Environmental Planner, to assess and document the risks and environmental impacts associated with the proposed project is generally all that is necessary to obtain enough information for the "Location Hydraulic Study". Any reasonable adaptation of the technical information for “Location Hydraulic Study” form, Figure 804.7A, may be utilized to document and summarize the findings of the "Location Hydraulic Study" when the project is expected to be processed with a categorical exclusion. Items listed in 23 CFR 650.111 as follows must be addressed:

(a) National Flood Insurance Program (NFIP) maps or information developed by the highway agency, if NFIP maps are not available, shall be used to determine whether a highway location alternative will include an encroachment.

(b) Location studies shall include evaluation and discussion of the practicability of alternatives to any longitudinal encroachments.

(c) Location studies shall include discussion of the following items, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base floodplain development:

(1) The risks associated with implementation of the action,

(2) The impacts on natural and beneficial floodplain values,

(3) The support of probable incompatible floodplain development,

(4) The measures to minimize floodplain impacts associated with the action, and

(5) The measures to restore and preserve the natural and beneficial floodplain values impacted by the action.

(d) Location studies shall include evaluation and discussion of the practicability of alternatives to any significant encroachments or any support of incompatible flood-plain development.

(e) The studies required by Sec. 650.111 (c) and (d) shall be summarized in environmental review documents prepared pursuant to 23 CFR part 771.

(f) Local, State, and Federal water resources and flood-plain management agencies should be consulted to determine if the proposed highway action is consistent with existing watershed and flood-plain management programs and to obtain current information on development and proposed actions in the affected watersheds.

Figure 804.7A is considered the suggested minimum hydraulic and engineering documentation for floodplain encroachments (bridge, culvert, channel change, slope protection, embankment, etc.). It is intended as a guide tool to help address the items listed in 23 CFR 650.111 and should be prepared jointly by the Project Engineer and Hydraulics Engineer. Since every location is unique, some of the questions may not apply, or additional considerations may need to be added.

For projects requiring an Environmental Impact Statement or Environmental Assessment (EIS/EA) or a finding of no significant impact (FONSI) with alternatives that have permanent features that encroach on the floodplain, a back-up report entitled Floodplain Evaluation is normally prepared by the District Environmental Branch. The technical requirements are typically developed jointly by the District Project Engineer and District Hydraulics Engineer. See Figure 804.7B for the Floodplain Evaluation Report Summary form that is used when an environmental document is to be prepared.

804.8 Design Standards

The design standards for highways encroaching on a floodplain are itemized in 23 CFR, Section 650.115. One requirement often overlooked is the need to assess the costs and risks associated with the overtopping flood for design alternatives in those instances where the overtopping flood exceeds the base flood. The content of design study information to be retained in the project file are described in 23 CFR, Section 650.117.

804.9 Coordination with the Local Community

The responsibility for enforcing National Flood Insurance Program (NFIP) regulations rests with the local community that is participating in the NFIP. It is the community who must submit proposals to Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community, or to demonstrate that an alternative floodway configuration meets NFIP requirements. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Therefore, the highway agency should deal directly with the community and, through them, deal with FEMA. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and study reports are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

Figure 804.7A

Technical Information for Location Hydraulic Study

Dist. _________ Co. _________ Rte._________ P.M. ________________
EA ___________________ Bridge No.__________________
Floodplain Description ____
____
____

1. Description of Proposal (include any physical barriers i.e. concrete barriers, soundwalls, etc. and design elements to minimize floodplain impacts)
   ____
   ____
   ____

2. ADT: Current Projected

3. Hydraulic Data: Base Flood Q_{100} =________ CFS
   WSE_{100} = ________ The flood of record, if greater than Q_{100}:
   Q =________ CFS WSE =________
   Overtopping flood Q =______ CFS WSE = ________
   Are NFIP maps available? Yes_____ No_____
   Are NFIP studies available? Yes_____ No_____

		Yes	No
4.	Is the highway location alternative within a regulatory floodway?	_______	_______
5.	Attach map with flood limits outlined showing all buildings or other improvements within the base floodplain. Potential Q_{100} backwater damages:		
	A. Residences?	_______	_______
	B. Other Bldgs?	_______	_______
	C. Crops?	_______	_______
	D. Natural and beneficial Floodplain values?	_______	_______
6.	Type of Traffic:		
	A. Emergency supply or evacuation route?	_______	_______
	B. Emergency vehicle access?	_______	_______
	C. Practicable detour available?	_______	_______
	D. School bus or mail route?	_______	_______

7. Estimated duration of traffic interruption for 100-year event ______ hours.

8. Estimated value of Q_{100} flood damages (if any) - moderate risk level.
   A. Roadway $________
   B. Property $________
   Total $________

9. Assessment of Level of Risk
   Low ___ Moderate ___ High ___
   For High Risk projects, during design phase, additional Design Study Risk Analysis may be necessary to determine design alternative.

PREPARED BY:

____ ____________________
Signature - Dist. Hydraulic Engineer Date
(Item numbers 3, 4, 5, 7, 9)

Is there any longitudinal encroachment, significant encroachment, or any support of incompatible Floodplain development? No ____ Yes ____

If yes, provide evaluation and discussion of practicability of alternatives in accordance with 23 CFR 650.113

Information developed to comply with the Federal requirement for the Location Hydraulic Study Shall be retained in the project files.

____ ____________________
Signature - Dist. Project Engineer Date
(Item numbers 1, 2, 6, 8)

FEMA has developed a comprehensive listing of all numerical models that are accepted for NFIP usage. These models can be accessed online at: http://www.fema.gov/mit/tsd/EN_modl.htm.

Topic 805 - Preliminary Plans

805.1 Required FHWA Approval

Current Federal policy requires the review and approval of plans for unusual structures. (See Indicies 805.2 - 805.6) by FHWA. FHWA will no longer review and approve major structures (those with greater than 125,000 square feet of deck area) or pumping plants with greater than 20 CFS design discharge. Submittal of plans for unusual structures for review applies only to new construction on the Interstate system. The responsibility for the oversight of unusual structures on other Federal-aid and non-Federal-aid highways will be assumed by the state.

Federal review and approval may take place at either their Division Office or FHWA Headquarters in Washington, D.C. Early submission of necessary data is critical in order to receive a timely approval.

805.2 Bridge Preliminary Report

A Bridge Preliminary Report will be prepared by Structures Design, in the Division of Engineering Services and submitted to the California FHWA Division Office in Sacramento for approval of unusual bridges and structures.

An unusual bridge involves difficult or unique foundation problems, new or complex designs involving unique design or operational features, longer than normal spans or bridges for which the design procedures depart from current acceptable practice. Examples include cable stayed, suspension, arch, segmental concrete bridges, trusses and other bridges which deviate from AASHTO Standard Specifications or Guide Specifications for Highway Bridges, bridges requiring abnormal dynamic analysis for seismic design, bridges designed using a three-dimensional computer analysis, bridges with spans exceeding 500 feet, and bridges which include ultra high strength concrete or steel.

805.3 Storm Drain Systems

The District will submit preliminary plans and hydraulic data for unusual storm drain systems to the California FHWA Division Office in Sacramento for storm drain systems that carry more than 200 CFS or have an accumulated surface detention storage system of more than five acre-feet.

805.4 Unusual Hydraulic Structures

The District will submit preliminary plans and hydraulic data for unusual hydraulic structures to the California FHWA Office in Sacramento. For projects on the interstate system, FHWA Headquarters Office of Bridge Technology approval is required for hydraulic structures involving unusual stream stability countermeasures or unique design techniques. The Division of Engineering Services will submit preliminary plans and hydraulic data to the California FHWA Division Office in Sacramento for unusual structures such as tunnels, complex or unique geotechnical structures and complex or unique hydraulic structures.

805.5 Levees and Dams Formed by Highway Fills

The District will submit preliminary plans and other supportive data to the California FHWA Division Office in Sacramento for approval of:

(a) Highway fills which will function as a levee and serve the purpose of reducing the flooding of adjacent areas.

(b) Dams formed by highway fills which will permanently impound water more than 25 feet in depth or 50 acre-feet in volume. See Index 829.9 Dams, for legal definition of a dam and regulations relative to approval by the California Department of Water Resources.

805.6 Geotechnical

The District shall submit preliminary plans and technical data for major or unusual geotechnical features to the California FHWA Division Office for approval. Major geotechnical features include unusually deep cuts or high fills where the site geology is potentially unstable, landslide corrections, and large retaining walls (cantilever, permanent ground anchor, and soil reinforcement). FHWA Headquarters Bridge Division approval is required for unusual geotechnical features, such as new or complex retaining wall systems or ground improvement systems.

805.7 Data Provided by the District

The following items of supportive information must be provided with requests for FHWA approval:

(a) Preliminary plans and profiles:

- Approach layouts.
- Drainage plans.

(b) Hydraulic design studies:

- Design Q and frequency.
- Hydraulic grade lines.
- Inflow - Outflow hydrographs.
- Capacity of reservoirs or pump storage systems.
- Pump capacity.
- Stream velocities.
- Water surface profiles.
- Slope protection, toe and top elevations.

(c) Proposed specifications.

(d) Estimated cost.

(e) Foundation report:

- Embankment design for fills functioning as dams.

(f) Subsurface investigations.

(g) Coordination with Federal, state and local agencies.

(h) Other pertinent data.

The FHWA requires that three copies of supportive information be submitted to the California FHWA Division Office when approval by FHWA Headquarters Bridge Division is required. Four copies of supportive information are to be furnished to the Division of Engineering Services to prepare the FHWA approval requests for bridges.

Topic 806 - Definitions of Drainage Terms

806.1 Introduction

These definitions are for use with Sections 800 through 890 of this manual and the references cited. They are not necessarily definitions as established by case or statutory law.

806.2 Drainage Terms

Accretion. Outward growth of bank or shore by sedimentation. Increase or extension of boundaries of land by action of natural forces.

Action. Any highway construction, reconstruction, rehabilitation, repair, or improvement.

Aggradation. General and progressive raising of a stream bed by deposition of sediment. Modification of the earth's surface in the direction of uniformity of grade, or slope, by deposition as in a river bed.

Aggressive. Refers to the corrosive properties of soil and water.

Alluvial. Referring to deposits of silts, sands, gravels and similar detrital material which have been transported by running water.

Alluvium. Stream-borne materials deposited in and along a channel.

Apron. (1) A paved area (usually depressed) around a drainage inlet. (2) A floor or lining of concrete between wingwalls at the end of a culvert to prevent scour. (3) A lining of the bed of the channel upstream or downstream from a lined or restricted waterway. (4) A floor or lining of concrete, rock, etc., to protect a surface from erosion such as the pavement along the toe of bank protection.

Aqueduct. (1) A major conduit. (2) The entire transmission main for a municipal water supply which may consist of a succession of canals, pipes, tunnels, etc. (3) Any conduit for water; especially one for a large quantity of flowing water. (4) A structure for conveying a canal over a river or hollow.

Aquifer. Water-bearing geologic formations that permit the movement of ground water.

Armor. Artificial surfacing of bed, banks, shore or embankment to resist erosion or scour.

Arroyo. Waterway of an ephemeral stream deeply carved in rock or ancient alluvium.

Artesian Waters. Percolating waters confined below impermeable formations with sufficient pressure to spring or well up to the surface.

Articulated. Made flexible by hinging particularly of small rigid slabs adapted to revetment.

Avulsion. (1) A forcible separation; also, a part torn off. (2) The sudden removal of land from the estate of one man to that of another, as by a sudden change in a river, the property thus separated continuing in the original owner. (3) A sudden shift in location of channel.

Backing Layer. A layer of graded rock between rock riprap and underlying engineering fabric or filter layer to prevent extrusion of the soil or filter layer material through the riprap.

Backshore. The zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water.

Backwater. An unnaturally high stage in stream caused by obstruction or confinement of flow, as by a dam, a bridge, or a culvert. Its measure is the excess of unnatural over natural stage, not the difference in stage upstream and downstream from its cause.

Baffle. Concrete or metal panels mounted in a series on the floor and/or wall of a culvert to increase boundary roughness and thereby reduce the average water velocity while increasing flow depth in the culvert.

Bank. The lateral boundary of a stream confining water flow. The bank on the left side of a channel looking downstream is called the left bank, etc.

Bankfull Stage. Stage at which a stream first overflows its natural banks into the floodplain. If the floodplain is absent or poorly defined, other indicators may identify bankfull. These include the height of depositional features, a change in vegetation, slope or topographic breaks along the bank, a change in the particle size of bank material, undercuts in the bank, and stain lines or the lower extent of lichens and moss on boulders. Corresponds to the stage at which channel maintenance is most effective, that is, the discharge at which the stream is moving sediment, forming or removing bars, forming or changing bends and meanders, and generally doing work that results in the average morphologic characteristics of channels. Generally applies to mature streams in more alluvial conditions rather than in mountainous conditions where the "bank" might be hundreds of feet above the incised channel. In incised channels, where the previous floodplain surface has become a terrace, the bankfull stage can be identified as the lowermost limit of establishing woody-riparian vegetation.

Bank Protection. Revetment, or other armor protecting a bank of a stream from erosion, includes devices used to deflect the forces of erosion away from the bank.

Bar. An elongated deposit of alluvium within a channel or across its mouth.

Barrier. A low dam or rack built to control flow of debris.

Base Flood. The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood). The "base flood" is commonly used as the "standard flood" in Federal flood insurance studies. (see Regulatory Flood).

Base Floodplain. The area subject to flooding by the base flood.

Basin. (1) The surface of the area tributary to a stream or lake. (2) Space above or below ground capable of retaining or detaining water or debris.

Bay. An indentation of bank or shore, including erosional cuts and slipouts, not necessarily large.

Beach. The zone of sedimentary material that extends landward from the low water line to the place where there is marked change in material or form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach, unless otherwise specified, is the mean low water line. A beach includes foreshore and backshore.

Bed. The earth below any body of water, limited laterally by bank or shore.

Bedding. The foundation under a drainage structure.

Bed Load. Sediment that moves by rolling, sliding, or skipping along the bed and is essentially in contact with the stream bed.

Berm. (1) A bench or terrace between two slopes. (2) A nearly horizontal part of the beach or backshore formed at the high water line by waves depositing material. Some beaches have no berms, others have one or several.

Block. Precast prismatic unit for riprap structure.

Bluff. A high, steep bank composed of erodible materials.

Boil. Turbulent break in a water surface by upwelling.

Boom. Floating log or similar element designed to dampen surface waves or control the movement of drift.

Bore. A transient solitary wave in a narrow or converging channel advancing with a steep turbulent front; product of flash floods or incoming tides.

Boulder. Largest rock transported by a stream or rolled in the surf; typically heavier than 25 pounds and larger than 8 inches in diameter.

Braided Stream. A stream in which flow is divided at normal stage by small islands. This type of stream has the aspect of a single large channel with which there are subordinate channels.

Breaker. A collapsing wave meeting a shore, reef, sandbar, or rock.

Breakwater. A fixed or floating structure that protects a shore area, harbor, anchorage, or basin from intercepting waves.

Bulkhead. A steep or vertical structure placed on a bank, bluff, or embankment to retain or prevent sliding of the land and protect the inland area from damage.

Bulking. The increase in volume of flow due to air entrainment, debris, bedload, or sediment in suspension.

Buoyancy. Uplift force on a submerged body equal to the mass of water displaced times the acceleration of gravity.

Camber. An upward adjustment of the profile of a drainage facility under a heavy loading (usually a high embankment) and poor soil conditions, so that as the drainage facility settles it approaches the design profile.

Canal. An artificial open channel.

Canyon. A large deep valley; also the submarine counterpart.

Cap. Top layer of stone protective works.

Capacity. The effective carrying ability of a drainage structure. Generally measured in cubic feet per second.

Capillarity. The attraction between water and soil particles which cause water to move in any direction through the soil mass regardless of gravitational forces.

Capillary Water. Water which clings to soil particles by capillary action. It is normally associated with fine sand, silt, or clay, but not normally with coarse sand and gravel.

Catch Basin. A drainage structure which collects water. May be either a structure where water enters from the side or through a grating.

Causeway. A raised embankment or trestle over swamp or overflow areas.

Cavitation. Erosion by suction, especially in the partial vacuum of a diverging jet.

Celerity. Velocity of a moving wave, as distinguished from velocity of particles oscillating in the wave.

Channel. An open conduit either naturally or artificially created which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided (see Braided Stream). Canal and "floodway" are some of the terms used to describe artificial channels.

Check. A sill or weir in a channel to control stage or velocity.

Check Dam. A small dam generally placed in steep ditches for the purpose of reducing the velocity in the ditch.

Cienega. A swamp formed by water rising to the surface at a fault.

Cleanout. An access opening to a roadway drainage system. Usually consists of a manhole shaft, a special chamber or opening into a shallow culvert or drain.

Cliff. A high, steep face of rock; a precipice.

Cloudburst. Rain storm of great intensity usually over a small area for a short duration.

Coast. (1) The strip of land, of indefinite width (up to several miles), that extends from the shoreline inland to the first major change in terrain features. (2) As a combining form, "upcoast" is northerly and "downcoast" is southerly.

Cobble. Rock smaller than a boulder and larger than gravel; typically 1 pound to 25 pounds, or 3 inches to 8 inches in diameter.

Coefficient of Runoff. Percentage of gross rainfall which appears as runoff.

Composite Hydrograph. A plot of mean daily discharges for a number of years of record on a single year time base for the purpose of showing the occurrence of high and low flows.

Concentrated Flow. Flowing water that has been accumulated into a single fairly narrow stream.

Concentration. In addition to its general sense, means the unnatural collection or convergence of waters so as to discharge in a narrower width, and at greater depth or velocity.

Conduit. Any pipe, arch, box or drain tile through which water is conveyed.

Cone. Physiographic form of sediment deposit washed from a gorge channel onto an open plain; a debris cone, also called an alluvial fan.

Confluence. A junction of streams.

Constriction. An obstruction narrowing a waterway.

Contraction. The reduction in cross sectional area of flow.

Control. (1) A section or reach of an open conduit or stream channel which maintains a stable relationship between stage and discharge. (2) For flood, erosion, debris, etc., remedial means or procedure restricting damage to a tolerable level.

Conveyance. A measure of the water carrying capacity of a stream or channel.

Core. Central zone of dike, levee, rock groin, jetty, etc.

Corrasion. Erosion or scour by abrasion in flowing water.

Corrosion. Erosion by chemical action.

Cradle. (1) A concrete base generally constructed to fit the shape of a structure which is to be forced through earthen material by a jacking operation. The cradle is constructed to line and grade. (2) Wood support for rigid culverts on yielding embankment subgrade. Then the pipe rides on the cradle as it is worked through the given material by jacking and tunneling methods. Also serves as bedding for pipes in trenches in special conditions.

Creek. A small stream, usually active.

Crest. (1) Peak of a wave or a flood. (2) Top of a levee, dam, weir, spillway or other water barrier or control.

Crib. An open-frame structure loaded with earth or stone ballast to act as a baffle in bank protection.

Critical Depth. (Depth at which specific energy is a minimum) - The depth of water in a conduit at which under certain other conditions the maximum flow will occur. These other conditions are the conduit is on the critical slope with the water flowing at its critical velocity and there is an adequate supply of water. The depth of water flowing in an open channel or a conduit partially filled, for which the velocity head equals one-half the hydraulic mean depth.

Critical Flow. That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of one.

Critical Slope. That slope at which the maximum flow will occur at the minimum velocity. The slope or grade that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; the slope of a conduit which will produce critical flow.

Critical Velocity. Mean velocity of flow when flow is at critical depth.

Culvert. A closed conduit which allows water to pass under a highway. The following three conditions constitute a culvert;

1. Single Barrel - span measured along centerline of road 20 feet or less.
2. Multi-Barrels - total of the individual spans measured along centerline of road is 20 feet or less.
3. Multi-Barrels - total of the individual spans measured along centerline of road is 20 feet or greater, but the distance between individual culverts is more than one-half the culvert diameter.

Current. Flow of water, both as a phenomenon and as a vector. Usually qualified by adjectives like downward, littoral, tidal, etc. to show relation to a pattern of movement.

Current Meter. An instrument for measuring the velocity of a current. It is usually operated by a wheel equipped with vanes or cups which is rotated by the action of the impinging current. An indicating or recording device is provided to indicate the speed of rotation which is correlated with the velocity of the current.

Cutoff Wall. A wall at the end of a drainage structure, the top of which is an integral part of the drainage structure. This wall is usually buried and its function is to prevent undermining of the drainage structure if the natural material at the outlet of the structure is dug out by the water discharging from the end of the structure. Cutoff walls are sometimes used at the upstream end of a structure when there is a possibility of erosion at this point.

Debris. Any material including floating woody materials and other trash, suspended sediment, or bed load moved by a flowing stream.

Debris Barrier. A deflector placed at the entrance of a culvert upstream, which tends to deflect heavy floating debris or boulders away from the culvert entrance during high-velocity flow.

Debris Basin. Any area upstream from a drainage structure utilized for the purpose of retaining debris in order to prevent clogging of drainage structures downstream.

Debris Rack. A straight barrier placed across the stream channel which tends to separate light and medium floating debris from stream flow and prevent the debris from reaching the culvert entrance.

Degradation. General and progressive lowering of the longitudinal profile of a channel by erosion.

Delta. System of channels thru an alluvial plain at the mouth of a stream.

Deposit. An earth mass of particles settled or stranded from moving water or wind.

Depth. Vertical distance, (1) from surface to bed of a body of water. (2) From crest or crown to invert of a conduit.

Design Capacity. The size required of a drainage facility which allows it to pass the design discharge without detrimental impacts.

Design Channel Capacity. Expressed as a rate of flow, usually in cubic feet per second, it is the level to which a facility is designed. Based upon slope, geometry, flow regime, frictional coefficients, etc., it is the sizing of a drainage facility which allows it to pass the design discharge. Freeboard or other safety factors which are added to the final facility dimensions are not a part of the design capacity.

Design Discharge. The quantity of flow that is expected at a certain point as a result of a design storm. Usually expressed as a rate of flow in cubic feet per second.

Design Flood. The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated by the design flood. In a FEMA floodplain, see 23 CFR, Part 650, Subpart A, for definitions of "overtopping flood" and "base flood."

Design Frequency. The recurrence interval for hydrologic events used for design purposes. As an example, a design frequency of 50 years means a storm of a magnitude that would be expected to recur on the average of every 50 years. (See Probability of Exceedance.)

Design High Water. The flood stage or tide crest elevation adopted for design of drainage and bank protection structures. (See Design Flood and High Water).

Design Storm. That particular storm which contributes runoff which the drainage facilities were designed to handle. This storm is selected for design on the basis of its probability of exceedance or average recurrence interval (See Probability of Exceedance.)

Detention Storage. Surface water moving over the land is in detention storage. Surface water allowed to temporarily accumulate in ponds, basins, reservoirs or other types of holding facility and which is ultimately returned to a watercourse or other drainage system as runoff is in detention storage. (See Retention Storage)

Detritus. Loose material such as; rock, sand, silt, and organic particles.

Dike. (1) Usually an earthen bank alongside and parallel with a river or open channel or an AC dike along the edge of a shoulder. (See Levee) (2) An AC dike along the edge of a shoulder.

Dike, Finger. Relatively short embankments constructed normal to a larger embankment, such as an approach fill to a bridge. Their purpose is to impede flow and direct it away from the major embankment.

Dike, Toe. Embankment constructed to prevent lateral flow from scouring the corner of the downstream side of an abutment embankment. Sometimes referred to as training dikes.

Dike, Training. Embankments constructed to provide a transition from the natural stream channel or floodplain, both to and from a constricting bridge crossing.

Discharge. A volume of water flowing out of a drainage structure or facility. Measured in cubic feet per second.

Dissipate. Expend or scatter harmlessly, as of energy of moving water.

Ditch. Small artificial channel, usually unlined.

Diversion. (1) The change in character, location, direction, or quantity of flow of a natural drainage course (a deflection of flood water is not a diversion). (2) Draft of water from one channel to another. (3) Interception of runoff by works which discharge it thru unnatural channels.

D-Load (Cracking D-Load). A term used in expressing the strength of concrete pipe. The cracking D-load represents the test load required to produce a 0.01 inch crack for a length of 12 inches.

Downdrain. A prefabricated drainage facility assembled and installed in the field for the purpose of transporting water down steep slopes.

Downdrift. The direction of predominant movement of littoral materials.

Drain. Conduit intercepting and discharging surplus ground or surface water.

Drainage. (1) The process of removing surplus ground or surface water by artificial means. (2) The system by which the waters of an area are removed. (3) The area from which waters are drained; a drainage basin.

Drainage Area (Drainage Basin) (Basin). That portion of the earth's surface upon which falling precipitation flows to a given location. With respect to a highway, this location may be either a culvert, the farthest point of a channel, or an inlet to a roadway drainage system.

Drainage Course. Any path along which water flows when acted upon by gravitational forces.

Drainage Divide. The rim of a drainage basin. A series of high points from which water flows in two directions, to the basin and away from the basin.

Drainage Easement (See Easement).

Drainage System. Usually a system of underground conduits and collector structures which flow to a single point of discharge.

Drawdown. The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and weirs or vertical spillways.

Drift. (1) Floating or non-mineral burden of a stream. (2) Deviation from a normal course in a cross current, as in littoral drift.

Drop. Controlled fall in a stream to dissipate energy.

Dry Weather Flows. A small amount of water which flows almost continually due to lawn watering, irrigation or springs.

Dune. A sand wave of approximately triangular cross section (in a vertical plane in the direction of flow) formed by moving water or wind, with gentle upstream slope and steep downstream slope and deposition on the downstream slope.

Easement. Right to use the land of others.

Ebb. Falling stage or outward flow, especially of tides.

Eddy. Rotational flow around a vertical axis.

Eddy Loss. The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

Embankment. Earth structure above natural ground.

Embayment. Indentation of bank or shore, particularly by progressive erosion.

Encroachment. Extending beyond the original, or customary limits, such as by occupancy of the river and/or flood plain by earth fill embankment.

Endwall. A wall placed at the end of a culvert. It may serve three purposes; (1), to hold the embankment away from the pipe and prevent sloughing into the pipe outlet channel; (2), to provide a wall which will prevent erosion of the roadway fill; and (3), to prevent flotation of the pipe.

Energy. Potential or kinetic, the latter being expressed in the same unit (feet) as the former.

Energy Dissipator. A structure for the purpose of slowing the flow of water and reducing the erosive forces present in any rapidly flowing body of water.

Energy Grade Line. The line which represents the total energy gradient along the channel. It is established by adding together the potential energy expressed as the water surface elevation referenced to a datum and the kinetic energy (usually expressed as velocity head) at points along the stream bed or channel floor.

Energy Head. The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the water in that section.

Entrance. The upstream approach transition to a constricted waterway.

Entrance Head. The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss. The head lost in eddies and friction at the inlet to a conduit or structure.

Ephemeral. Of brief duration, as the flow of a stream in an arid region.

Equalizer. A drainage structure similar to a culvert but different in that it is not intended to pass a design flow in a given direction. Instead it is often placed level so as to permit passage of water in either direction. It is used where there is no place for the water to go. Its purpose is to maintain the same water surface elevation on both sides of the highway embankment.

Erosion. The wearing away of natural (earth) and unnatural (embankment, slope protection, structure, etc.) surfaces by the action of natural forces, particularly moving water and materials carried by it. In the case of drainage terminology, this term generally refers to the wearing away of the earth's surface by flowing water.

Erosion and *Scour*. The cutting or wearing away by the forces of water of the banks and bed of a channel in horizontal and vertical directions, respectively.

Erosion and *Accretion*. Loss and gain of land, respectively, by the gradual action of a stream in shifting its channel by cutting one bank while it builds on the opposite bank. Property is lost by erosion and gained by accretion but not by *avulsion* when the shift from one channel to another is sudden. Property is gained by *reliction* when a lake recedes.

Estuary. That portion of a river channel occupied at times or in part by both sea and river flow in appreciable quantities. The water usually has brackish characteristics.

Evaporation. A process whereby water as a liquid is changed into water vapor, typically through heat supplied from the sun.

Face. The outer layer of slope revetment.

Fan. A portion of a cone, but sometimes used to emphasize definition of radial channels. Also reference to spreading out of water or soils associated with waters leaving a confined channel (e.g., alluvial fan).

Fetch. The unobstructed distance across open water through which wind acts to generate waves.

Filter. A porous article or mass (as of fabric or even-graded mineral aggregate) through which water will freely pass, but which will block the passage of soil particles.

Filter Fabric (RSP fabric). An engineering fabric (geotextile) placed between the backfill and supporting or underlying soil through which water will pass and soil particles are retained.

Filter Layer. A layer of even-graded rock between rock riprap and underlying soil to prevent extrusion of the soil thru riprap.

Flap Gate. This is a form of valve that is designed so that a minimum force is required to push it open but when a greater water pressure is present on the outside of the valve, it remains shut so as to prevent water from flowing in the wrong direction. Construction is simple with a metal cover hanging from an overhead rod or pinion at the end of a culvert or drain.

Flood Frequency. Also referred to as exceedance interval, recurrence interval or return period; the average time interval between actual occurrences of a hydrological event of a given or greater magnitude; the percent chance of occurrence is the reciprocal of flood frequency, e.g., a 2 percent chance of occurrence is the reciprocal statement of a 50-year flood. (See Probability of Exceedance.)

Floodplain. Normally dry land areas subject to periodic temporary inundation by stream flow or tidal overflow. Land formed by deposition of sediment by water; alluvial land.

Floodplain Encroachment. An action within the limits of the base flood plain.

Flood Plane. The position occupied by the water surface of a stream during a particular flood. Also, loosely, the elevation of the water surface at various points along the stream during a particular flood.

Floodproof. To design and construct individual buildings, facilities, and their sites to protect against structural failure, to keep water out or reduce the effects of water entry.

Flood Stage. The elevation at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured. The elevation of the lowest bank of the reach. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area.

Flood Waters. Former stream waters which have escaped from a watercourse (and its overflow channel) and flow or stand over adjoining lands. They remain as such until they disappear from the surface by infiltration, evaporation, or return to a natural watercourse. They do not become surface waters by mingling with such waters, nor stream waters by eroding a temporary channel.

Flow. A term used to define the movement of water, silt, sand, etc.; discharge; total quantity carried by a stream.

Flow Line. A term used to describe the line connecting the low points in a watercourse.

Flow Regime. The system or order characteristic of streamflow with respect to velocity, depth, and specific energy.

Flow, steady. Flow at constant discharge.

Flow, unsteady. Flow on rising or falling stages.

Flow, varied. Flow in a channel with variable section.

Foreshore. The part of the shore lying between the ordinary high water mark or upper limit of wave wash traversed by the runup and return of waves and the water's edge at the low water.

Freeboard. (1) The vertical distance between the water surface elevation usually corresponding to the design flow and a point of interest such as a bridge beam, levee top or specific location on the roadway grade. (2) The distance between the normal operating level and the top of the sides of an open conduit; the crest of a dam, etc., designed to allow for wave action, superelevation, floating debris, or any other condition or emergency, without overtopping the structure. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors such as the accumulation of silt, trash, or aquatic growth in the channel; unforeseen embankment settlement, erratic hydrologic phenomena and variation of resistance or other coefficients from those assumed in design.

Free Outlet. A condition under which water discharges with no interference such as a pipe discharging into open air.

Free Water. Water which can move through the soil by force of gravity.

French Drain. A trench loosely backfilled with stones, the largest stones being placed in the bottom with the size of stones decreasing towards the top. The interstices between the stones serve as a passageway for water.

Friction. Energy-dissipating conflict among turbulent water particles disturbed by irregularities of channel surface.

Froude Number. A dimensionless expression of the ratio of inertia forces to gravity forces, used as an index to characterize the type of flow in a hydraulic structure in which gravity is the force producing motion and inertia is the resisting force. It is equal to a characteristic flow velocity (mean, surface, or maximum) of the system divided by the square root of the product of a characteristic dimension (as diameter of depth) and the gravity constant (acceleration due to gravity) all expressed in consistent units. $F_r = V/(gy)^{1/2}$

Gabion. A wire basket or cage filled with stone and placed as, or as part of, a bank-protection structure.

Gaging Station. A location on a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is uniform, a control downstream from this reach and usually a small building to house the recording instruments.

Gorge. A narrow deep valley with steep or vertical banks.

Grade. Elevation of bed or invert of a channel.

Grade to Drain. A construction note often inserted on a plan for the purpose of directing the Contractor to slope a certain area in a specific direction, so that the surface waters will flow to a designated location.

Gradient (Slope). The rate of ascent or descent expressed as a percent or as a decimal as determined by the ratio of the change in elevation to the length.

Gradually Varied Flow. In this type of flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces are neglected.

Gravel. Rock larger than sand and smaller than cobble, arbitrarily ranging in diameter from 0.2 inch to 2 inches.

Groin. A fingerlike barrier structure usually built perpendicular to the shoreline or oblique to primary motion of water, to trap littoral drift, retard erosion of the shore, or to control movement of bed material.

Ground Water. That water which is present under the earth's surface. Ground water is that situated below the surface of the land, irrespective of its source and transient status. Subterranean streams are flows of ground waters parallel to and adjoining stream waters, and usually determined to be integral parts of the visible streams.

Grouted. Bonded together with an inlay or overlay of cement mortar.

Guide Bank. An appendage to the highway embankment at or near a bridge abutment to guide the stream through the bridge opening.

Gulch. A relatively young, well-defined and sharply cut erosional channel.

Gully. Diminutive of gulch.

Head. Represents an available force equivalent to a certain depth of water. This is the motivating force in effecting the movement of water. The height of water above any point or plane of reference. Used also in various compound expressions, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc.

Headcutting. Progressive scouring and degrading of a streambed at a relatively rapid rate in the upstream direction, usually characterized by one or a series of vertical falls.

High Water. Maximum flood stage of stream or lake; periodic crest stage of tide. Historic HW is stage recorded or otherwise known.

Hydraulic. Pertaining to water in motion and the mechanics of the motion.

Hydraulic Gradient. A line which represents the relative force available due to the potential energy available. This is a combination of energy due to the height of the water and the internal pressure. In any open channel, this line corresponds to the water surface. In a closed conduit, if several openings were placed along the top of the pipe and open tubes inserted, a line connecting the water surface in each of these tubes would represent the hydraulic grade line.

Hydraulic Jump (or Jump). Transition of flow from the rapid to the tranquil state. A varied flow phenomenon producing a rise in elevation of water surface. A sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy.

Hydraulic Mean Depth. The area of the flow cross section divided by the water surface width.

Hydraulic Radius. The cross sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.

Hydrograph. A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrographic. Pertaining to the measurement or study of bodies of water and associated terrain.

Hydrography. Water Surveys. The art of measuring, recording, and analyzing the flow of water; and of measuring and mapping watercourses, shore lines, and navigable waters.

Hydrologic. Pertaining to the cyclic phenomena of waters of the earth; successively as precipitation, runoff, storage and evaporation, and quantitatively as to distribution and concentration.

Hydrology. The science dealing with the occurrence and movement of water upon and beneath the land areas of the earth. Overlaps and includes portions of other sciences such as meteorology and geology. The particular branch of Hydrology that a design engineer is generally interested in is surface runoff which is the result of excess precipitation.

Hydrostatic. Pertaining to pressure by and within water due to gravitation acting thru depth.

Hyetograph. Graphical representation of rainfall intensity against time.

Impinge. To strike and attack directly, as in curvilinear flow where the current does not follow the curve but continues on tangent into the bank on the outside of bend in the channel.

Incised Channel. Those channels which have been cut relatively deep into underlying formations by natural processes. Characteristics include relatively straight alignment and high, steep banks such that overflow rarely occurs, if ever.

Infiltration. The passage of water through the soil surface into the ground.

Inlet Time. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point where it enters a drain or culvert.

Inlet Transition. A specially shaped entrance to a box or pipe culvert. It is shaped in such a manner that in passing from one flow condition to another, the minimum turbulence or interference with flow is permitted.

Inundate. To cover with a flood.

Invert. The bottom of a drainage facility along which the lowest flows would pass.

Invert Paving. Generally applies to metal pipes where it is desirable to improve flow characteristics or prevent corrosion at low flows. The bottom portion of the pipe is paved with an asphaltic material, concrete, or air-blown mortar.

Inverted Siphon. A pipe for conducting water beneath a depressed place. A true inverted siphon is a culvert which has the middle portion at a lower elevation than either the inlet or the outlet and in which a vacuum is created at some point in the pipe. A sag culvert is similar, but the vacuum is not essential to its operation.

Isohyetal Line. A line drawn on a map or chart joining points that receive the same amount of precipitation.

Isohyetal Map. A map containing isohyetal lines and showing rainfall intensities.

Isovel. Line on a diagram of a channel connecting points of equal velocity.

Jack (or Jack Straw). Bank protection element consisting of wire or cable strung on three mutually perpendicular struts connected at their centers.

Jacking Operations. A means of constructing a pipeline under a highway without open excavation. A cutting edge is placed on the first section of pipe and the pipe is forced ahead by hydraulic jacks. As the leading edge pushes ahead, the material inside the pipe is dug out and transported outside the pipe for disposal.

Jam. Wedged collection of drift in a constriction of a channel, such as a gorge or a bridge opening.

Jet. An effluent stream from a restricted channel, including a fast current through a slower stream.

Jetty. An elongated, artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection of strength of currents and waves.

Jump. Sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy; the hydraulic jump.

Kolk. Rotational flow about a horizontal axis, induced by a reef and breaking the surface in a boil.

Lake. A water filled basin with restricted or no outlet. Includes reservoirs, tidal ponds and playas.

Lag. Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

Laminar Flow. That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the velocity (as opposed to turbulent flow).

Lateral. In a roadway drainage system, a drainage conduit transporting water from inlet points to the main drain trunk line.

Levee. An embankment on or along the bank of a stream or lake to protect outer lowlands from inundation. (See Dike)

Lining. Protective cover of the perimeter of a channel.

Littoral. Pertaining to or along the shore, particularly to describe currents, deposits, and drift.

Littoral Drift. The sedimentary material (sand) moved along the shoreline under the influence of waves and currents.

Littoral Transport. The movement of littoral drift along the shoreline by waves and currents. Includes movement parallel (longshore transport) and perpendicular (on-offshore transport) to the shore.

Local Depression. A low area in the pavement or in the gutter established for the special purpose of collecting surface waters on a street and directing these waters into a drainage inlet.

Longshore. Parallel to and near the shoreline.

Marginal. Within a borderland area; more general and extensive than riparian.

Marsh. An area of soft, wet, or periodically submerged land, generally treeless and usually characterized by grasses and other low vegetation.

Mature. Classification for streams which have established flat gradients not subject to further scour.

Maximum Historical Flood. The maximum flood that has been recorded or experienced at any particular highway location.

Mean Annual Flood. The flood discharge with a recurrence interval of 2.33 years.

Mean Depth. For a stream at any stage, the wetted normal section divided by the surface width. Hydraulic mean depth.

Meander. In connection with streams, a winding channel usually in an erodible, alluvial valley. A reverse or S-shaped curve or series of curves formed by erosion of the concave bank, especially at the downstream end, characterized by curved flow and alternating shoals and bank erosions. Meandering is a stage in the migratory movement of the channel, as a whole, down the valley.

Meander Plug (Clay Plug). Deposits of cohesive materials in old channel bendways. These plugs are sufficiently resistant to erosion to serve as essentially semi-permanent geological controls to advancing channel migrations.

Meander Scroll. Evidence of historical meander patterns in the form of lines visible on the inside of meander bends (particularly on aerial photographs) which resemble a spiral or convoluted form in ornamental design. These lines are concentric and regular forms in high sinuosity channels and are largely absent in poorly developed braided channels.

Mesh. Woven wire or other filaments used alone as revetment, or as retainer or container of masses of gravel or cobble.

Mud Flow. A well-mixed mass of water and alluvium which, because of its high viscosity, and low fluidity as compared with water, moves at a much slower rate, usually piling up and spreading out like a sheet of wet mortar or concrete.

Natural and Beneficial Floodplain Values. Includes but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

Natural Channel Capacity. The maximum rate of flow in cubic feet per second that can pass through a channel without overflowing the banks

Navigable Waters. Those stream waters lawfully declared or actually used as such. Navigable Waters of the State of California are those declared by Statute. Navigable Waters of the United States are those determined by the Corps of Engineers or the U.S. Coast Guard to be so used in interstate or international commerce. Other streams have been held navigable by courts under the common law that navigability in fact is navigability in law.

Negative Projecting Conduits. A structure installed in a trench with the top below the top of trench, then covered with backfill and embankment. See Positive Projecting Conduit

Nonuniform Flow. A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc.

Normal Depth. The depth at which flow is steady and hydraulic characteristics are uniform.

Normal Water Surface (Natural Water Surface). The free surface associated with flow in natural streams.

"n" Value. The roughness coefficient in the Manning formula for determination of the discharge coefficient in the Chezy formula,

$$V = C(RS)^{1/2}, \text{where } C = \left(\frac{1.49}{n}\right)R^{1/6}$$

Nourishment. The process of replenishing a beach. It may be brought about naturally, by accretion due to the longshore transport, or artificially, by the deposition of dredged materials.

Off-Site Drainage. The handling of that water which originates outside the highway right of way.

On-Site Drainage. The handling of that water which originates inside the highway right of way.

Open Channel. Any conveyance in which water flows with a free surface.

Ordinary High Water Mark. The line on the shore established by the fluctuation of water and physically indicated on the bank (1.5 $\pm$ years return period)

Outfall. Discharge or point of discharge of a culvert or other closed conduit.

Outwash. Debris transported from a restricted channel to an unrestricted area where it is deposited to form an alluvial or debris cone or fan.

Overflow. Discharge of a stream outside its banks; the parallel channels carrying such discharge.

Overtopping Flood. The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

Peak Flow. Maximum momentary stage or discharge of a stream in flood. Design Discharge.

Pebble. Stone 0.5 inch to 3-inch in diameter, including coarse gravel and small cobble.

Perched Water. Ground water located above the level of the water table and separated from it by a zone of impermeable material.

Percolating Waters. Waters which have infiltrated the surface of the land and move slowly downward and outward through devious channels (aquifers) unrelated to stream waters, until they reach an underground lake or regain and spring from the land surface at a lower point.

Permeability. The property of soils which permits the passage of any fluid. Permeability depends on grain size, void ratio, shape and arrangement of pores.

Permeable. Open to the passage of fluids, as for (1) pervious soils and (2) bank-protection structures.

Physiographic Region. A geographic area whose pattern of landforms differ significantly from that of adjacent regions.

Pier. Vertical support of a structure standing in a stream or other body of water. Used in a general sense to include bents and abutments.

Pile. A long, heavy timber or section of concrete or metal that is driven or jetted into the earth or bottom of a water body to serve as a structural support or protection.

Piping. The action of water passing through or under an embankment and carrying some of the finer material with it to the surface at the downstream face.

Plunge. Flow with a strong downward component, as in outfall drops, overbank falls, and surf attack on a beach.

Point of Concentration. That point at which the water flowing from a given drainage area concentrates. With reference to a highway, this would generally be either a culvert entrance or some point in a roadway drainage system.

Poised Stream. A term used by river engineers applying to a stream that over a period of time is neither degrading or aggrading its channel, and is nearly in equilibrium as to sediment transport and supply.

Positive Projecting Conduit. A structure installed in shallow trench with the top of the conduit projecting above the top of the trench and then covered with embankment. See Negative Projecting Conduit.

Potamology. The hydrology of streams.

Practicable. Capable of being done within reasonable natural, social, and economic constraints.

Precipitation. Discharge of atmospheric moisture as rain, snow or hail, measured in depth of fall or in terms of intensity of fall in unit time.

Prescriptive Rights. The operation of the law whereby rights may be established by long exercise of their corresponding powers or extinguished by prolonged failure to exercise such powers.

Preserve. To avoid modification to the functions of the natural floodplain environment or to maintain it, as closely as practicable, in its natural state.

Probability. The chance of occurrence or recurrence of a specified event within a unit of time, commonly expressed in 3 ways. Thus a 10-year flood has a chance of 0.1 per year and is also called a 10 percent-chance flood.

Probability of Exceedance. The statistical probability, expressed as a percentage, of a hydrologic event occurring or being exceeded in any given year. The probability (p) of a storm or flood is the reciprocal of the average recurrence interval (N).

Probable Maximum Flood. The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region.

Pumping Plant. A complete pumping installation including a storage box, pump or pumps, standby pumps, connecting pipes, electrical equipment, pumphouse and outlet chamber.

Rack. An open upright structure, such as a debris rack.

Rainfall. Point Precipitation: That which registers at a single gauge. Area Precipitation: Adjusted point rainfall for area size.

Rainwash. The creep of soil lubricated by rain.

Range. Difference between extremes, as for stream or tide stage.

Rapidly Varied Flow. In this type of flow, changes in depth and velocity take place over short distances, acceleration forces dominate, and energy loss due to friction is minor.

Rapids. Swift turbulent flow in a rough steep reach.

Reach. The length of a channel uniform with respect to discharge, depth, area, and slope. More generally, any length of a river or drainage course.

Recession. Retreat of shore or bank by progressive erosion.

Reef. Generally, any solid projection from the bed of a stream or other body of water.

Regime. The system or order characteristic of a stream; its behavior with respect to velocity and volume, form of and changes in channel, capacity to transport sediment, amount of material supplied for transportation, etc.

Regimen. The characteristic behavior of a stream during ordinary cycles of flow.

Regulatory Floodway. The open floodplain area that is reserved in by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP)).

Reliction. Pertaining to being left behind. For example: that area of land is left behind by reliction when the water surface of a lake is lowered.

Repose. The stable slope of a bank or embankment, expressed as an angle or the ratio of horizontal to vertical projection.

Restore. To reestablish a setting or environment in which the functions of the natural and beneficial floodplain values adversely impacted by the highway agency can continue to operate.

Restriction. Artificial or natural control against widening of a channel, with or without construction.

Retard. Bank-protection structure designed to check the riparian velocity and induce silting or accretion.

Retarding Basin. Either a natural or man made basin with the specific function of delaying the flow of water from one point to another. This tends to increase the time that it takes all the water falling on the extremities of the drainage basin to reach a common point, resulting in a reduced peak flow at that point.

Retention Storage. Water which accumulates and ponds in natural or excavated depressions in the soil surface with no possibility for escape as runoff. (See Detention Storage)

Retrogression. Reversal of stream grading; i.e., aggradation after degradation, or vice versa.

Revetment. Bank protection to prevent erosion.

Riparian. Pertaining to the banks of a stream.

Riprap. A layer, facing, or protective mound of rubble or stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also, the stone used for this purpose.

Ripple. (1) The light fretting or ruffling of a water caused by a breeze. (2) Undulating ridges and furrows, or crests and troughs formed by action of the flow.

Risk. The consequences associated with the probability of flooding attributable to an encroachment. It includes the potential for property loss and hazard to life during the service life of the highway.

Risk Analysis. An economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least expected cost to the public. It must include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway aggravated flood damage to other property, and for additional or interrupted highway travel.

Riser. In mountainous terrain where much debris is encountered, the entrance to a culvert sometimes becomes easily clogged. Therefore, a corrugated metal pipe or a structure made of timber or concrete with small perforations, called a riser, is installed vertically to permit entry of water and prohibit the entry of mud and debris. The riser may be increased in height as the need occurs.

River. A large stream, usually active when any streams are flowing in the region.

Rock. (1) Cobble, boulder or quarry stone as a construction material. (2) Hard natural mineral, in formation as in piles of talus.

Rounded Inlet. The edges of a culvert entrance that are rounded for smooth transition which reduces turbulence and increases capacity.

RSP Fabric. (See Filter Fabric).

Rubble. Rough, irregular fragments of rock or concrete.

Runoff. (1) The surface waters that exceed the soil's infiltration rate and depression storage. (2) The portion of precipitation that appears as flow in streams. Drainage or flood discharge which leaves an area as surface flow or a pipeline flow, having reached a channel or pipeline by either surface or subsurface routes.

Runup. The rush of water up a beach or structure, associated with the breaking of a wave. The amount of runup is measured according to the vertical height above still water level that the rush of water reaches.

Sag Culvert (or Sag Pipe). A pipeline with a dip in its grade line crossing over a depression or under a highway, railroad, canal, etc. The term inverted siphon is common but inappropriate as no siphonic action is involved. The term "sag pipe" is suggested as a substitute.

Sand. Granular soil coarser than silt and finer than gravel, ranging in diameter from 0.002 inch to 0.2 inch.

Scour. The result of erosive action of running water, primarily in streams, excavating and carrying away material from the bed and banks. Wearing away by abrasive action.

Scour, General. The removal of material from the bed and banks across all or most of the width of a channel, as a result of a flow contraction which causes increased velocities and bed shear stress.

Scour, Local. Removal of material from the channel bed or banks which is restricted to a minor part of the width of a channel. This scour occurs around piers and embankments and is caused by the actions of vortex systems induced by the obstruction to the flow.

Scour, Natural. Removal of material from the channel bed or banks which occurs in streams with the migration of bed forms, shifting of the thalweg and at bends and natural contractions.

Sea. Ocean or other body of water larger than a lake; state of agitation of any large body of water.

Seawall. A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. (See bulkhead).

Sediment. Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited by water.

Sedimentation. Gravitational deposit of transported material in flowing or standing water.

Seepage. Percolation of underground water thru the banks and into a stream or other body of water.

Seiche. A standing wave oscillation of an enclosed waterbody that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric.

Seismic Wave. A gravity wave caused by an earthquake.

Sheet Flow. Any flow spread out and not confined; i.e., flow across a flat open field.

Sheet Pile. A pile with a generally slender, flat cross-section that is driven into ground or bottom of a water body and meshed or interlocked with like members to form a wall or bulkhead.

Shoal. A shallow region in flowing or standing water, especially if made shallow by deposition.

Shoaling. Deposition of alluvial material resulting in areas with relatively shallow depth.

Shore. The narrow strip of land in immediate contact with the water, including the zone between high and low water lines. See backshore, foreshore, onshore, offshore, longshore, and nearshore.

Significant Encroachment. A highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction or flood related impacts:

- A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
- A significant risk, or
- A significant adverse impact on natural and beneficial floodplain values.

Silt. (1) *Water-Borne Sediment.* Detritus carried in suspension or deposited by flowing water, ranging in diameter from 0.0002 inch to 0.002 inch. The term is generally confined to fine earth, sand, or mud, but is sometimes both suspended and bedload. (2) *Deposits of Water-Borne Material.* As in a reservoir, on a delta, or on floodplains.

Sinuosity. The ratio of the length of the river thalweg to the length of the valley proper.

Skew. When a drainage structure is not normal (perpendicular) to the longitudinal axis of the highway, it is said to be on a skew. The skew angle is the smallest angle between the perpendicular and the axis of the structure.

Slide. Gravitational movement of an unstable mass of earth from its natural position.

Slipout. Gravitational movement of an unstable mass of earth from its constructed position. Applied to embankments and other man-made earthworks.

Slope. (1) Gradient of a stream. (2) Inclination of the face of an embankment, expressed as the ratio of horizontal to vertical projection; or (3) The face of an inclined embankment or cut slope. In hydraulics it is expressed as percent or in decimal form.

Slough. (1) Pronounced SLU. A side or overflow channel in which water is continually present. It is stagnant or slack; also a waterway in a tidal marsh. (2) Pronounced SLUFF. Slide or slipout of a thin mantle of earth, especially in a series of small movements.

Slugflow. Flow in culvert or drainage structure which alternates between full and partly full. Pulsating flow -- mixed water and air.

Soffit. The bottom of the top -- (1) With reference to a bridge, the low point on the underside of the suspended portion of the structure. (2) In a culvert, the uppermost point on the inside of the structure.

Specific Energy. The energy contained in a stream of water, expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity.

Spur Dike. A structure or embankment projecting a short distance into a stream from the bank and at an angle to deflect flowing water away from critical areas.

Stage. The elevation of a water surface above its minimum; also above or below an established "low water" plane; hence above or below any datum of reference; gage height.

Standing Wave. The motion of swiftly flowing stream water, that resembles a wave, but is formed by decelerating or diverging flow that does not quite produce a hydraulic jump. A term which when used to describe the upper flow regime in alluvial channels, means a vertical oscillation of the water surface between fixed nodes without appreciable progression in either an upstream or downstream direction. To maintain the fixed position, the wave must have a celerity (velocity) equal to the approach velocity in the channel, but in the opposite direction.

Steady Flow. A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant.

Stone. Rock or rock-like material; a particle of such material, in any size from pebble to the largest quarried blocks.

Storage. Detention, or retention of water for future flow, naturally in channel and marginal soils or artificially in reservoirs.

Storage Basin. Space for detention or retention of water for future flow, naturally in channel and marginal soils, or artificially in reservoirs.

Storm. A disturbance of the ordinary, average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Storm Drain. That portion of a drainage system expressly for collecting and conveying former surface water in an enclosed conduit. Often referred to as a "storm sewer", storm drains include inlet structures, conduit, junctions, manholes, outfalls and other appurtenances.

Storm Water Management. The recognition of adverse drainage resulting from altered runoff and the solutions resulting from the cooperative efforts of public agencies and the private sector to mitigate, abate, or reverse those adverse results.

Strand. (1) To lodge on bars, banks, or overflow plain, as for drift. (2) Bar of sediment connecting two regions of higher ground.

Stream. Water flowing in a channel or conduit, ranging in size from small creeks to large rivers.

Stream Power. An expression used in predicting bed forms and hence bed load transport in alluvial channels. It is the product of the mean velocity, the specific weight of the water-sediment mixture, the normal depth of flow and the slope.

Stream Response. Changes in the dynamic equilibrium of a stream by any one, or combination of various causes.

Stream Waters. Former surface waters which have entered and now flow in a well defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in bed or banks of the watercourse. They continue as stream waters as long as they flow in the watercourse, including overflow and multiple channels as well as the ordinary or low-water channel.

Strutting. Elongation of the vertical axis of pipe prior to installing in a trench. After the backfill has been placed around the pipe and compacted, the wires or rods holding the pipe in its distorted shape are removed. Greater side support from the earth is developed when the pipe tends to return to its original shape. Generally used on pipes which because of size or thinness of the metal would tend to deform during construction operations. Arches are strutted diagonally per standard or special plan.

Subcritical Flow. In this state, gravity forces are dominant, so that the flow has a low velocity and is often described as tranquil and streaming. Also, that flow which has a Froude number less than one.

Subdrain. A conduit for collecting and disposing of underground water. It generally consists of a pipe, with perforations in the bottom through which water can enter.

Subsidence. General lowering of land surface by consolidation or removal of underlying soil.

Sump. In drainage, any low area which does not permit the escape of water by gravity flow.

Supercritical Flow. In this state, inertia forces are dominant, so that flow has a high velocity and is usually described as rapid, shooting and torrential. Also, that flow which has a Froude number greater than one.

Support Base Floodplain Development. To encourage, allow, serve, or otherwise facilitate additional base floodplain development. Direct support results from an encroachment, while indirect support results from an action out of the base floodplain.

Surf. The breaking of waves and swell on the foreshore and offshore shoals.

Surface Runoff. The movement of water on earth's surface, whether flow is over surface of ground or in channels.

Surface Waters. Surface waters are those which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel. They appear as puddles, sheet or overland flow, and rills, and continue to be surface waters until they disappear from the surface by infiltration or evaporation, or until by overland or vagrant flow they reach well-defined watercourses or standing bodies of water like lakes or seas.

Surge. A sudden swelling of discharge in unsteady flow.

Suspended Load. Sediment that is supported by the upward components of turbulent currents in a stream and that stay in suspension for appreciable amount of time.

Swale. A shallow, gentle depression in the earth's surface. This tends to collect the waters to some extent and is considered in a sense as a drainage course, although waters in a swale are not considered stream waters.

Swamp. An area of shallow pondage or saturated surface, the water being fresh or acidic and the area usually covered with rank vegetation.

Swell. Waves generated by a distant storm, usually regular and fully harmonic.

Talus. Loose rocks and debris disintegrated from a steep hill or cliff standing at repose along the toe.

Tapered Inlet. A transition to direct the flow of water into a channel or culvert. A smooth transition to increase hydraulic efficiency of an inlet structure.

Terrace. Berm or bench-like earth embankment, with a nearly level plain bounded by rising and falling slopes.

Tetrahedron. Bank protection element, basically composed of 6 steel or concrete struts joined like the edges of a triangular pyramid, together with subdividing struts and tie wires or cables.

Tetrapod. Bank protection element, precast of concrete, consisting of 4 legs joined at a central block, each leg making an angle of 109.5 degrees with the other three, like rays from the center of a tetrahedron to the center of each face.

Texture. Arrangement and interconnection of surface and near-surface particles of terrain or channel perimeter.

Thalweg. The line following the lowest part of a valley, whether under water or not. Usually the line following the deepest part of the bed or channel of a river.

Thread. The central element of a current, continuous along a stream.

Tide. The periodic rising and falling of the ocean and connecting bodies of water that results from gravitational attraction of the moon and sun acting on the rotating earth.

Time of Concentration. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point under consideration. It is usually associated with the design storm.

Topping. The top layer on horizontal revetments or rock structures; also capping or cap stones.

Training. Control of direction of currents.

Transition. A relatively short reach or conduit leading from one waterway section to another of different width, shape, or slope.

Transport. To carry solid material in a stream in solution, suspension, saltation, or entrainment.

Trash Rack. A grid or screen across a stream designed to catch floating debris.

Trough. Space between wave crests and the water surface below it.

Trunk (or Trunk Line). In a roadway drainage system, the main conduit for transporting the storm waters. This main line is generally quite deep in the ground so that laterals coming from fairly long distances can drain by gravity into the trunk line.

Tsunami. A gravity wave caused by an underwater seismic disturbance (such as sudden faulting, landsliding or volcanic activity).

Turbulence. A state of flow wherein the water is agitated by cross-currents and eddies, as opposed to a condition of flow that is quiet and laminar.

Turbulent Flow. That type of flow in which any particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the square of the velocity.

Undercut. Erosion of the low part of a steep bank so as to compromise stability of the upper part.

Underflow. The downstream flow of water through the permeable deposits that underlie a stream. (1) Movement of water through a pervious subsurface stratum, the flow of percolating water; or water under ice, or under a structure. (2) The rate of flow or discharge of subsurface water.

Undertow. Current outward from a wave-swept shore carrying solid particles swept or scoured from the beach or foreshore.

Unsteady Flow. A flow in which the velocity changes with respect to space and time.

Updrift. The direction opposite that of the predominant movement of littoral materials.

Uplift. Upward hydrostatic pressure on base of an impervious structure.

Velocity. The rate of motion of objects or particles, or of a stream of particles.

Velocity Head. A term used in hydraulics to represent the kinetic energy of flowing water. This "head" is represented by a column of standing water equivalent in potential energy to the kinetic energy of the moving water calculated as ($V^2/2g$) where the "V" represents the velocity in feet per second and "g" represents the potential acceleration due to gravity, in feet per second per second.

Vernal Pools. Seasonally flooded landscape depressions that support distinctive (and many times rare) plant and animal species adapted to periodic or continuous inundation during the wet season, and the absence of either ponded water or wet soil during the dry season.

Wash. Flood plain or active channel of an ephemeral stream, usually in recent alluvium.

Watercourse. A definite channel with bed and banks within which water flows, either continuously or in season. A watercourse is continuous in the direction of flow and may extend laterally beyond the definite banks to include overflow channels contiguous to the ordinary channel. The term does not include artificial channels such as canals and drains, except natural channels trained or restrained by the works of man. Neither does it include depressions or swales through which surface or errant waters pass.

Watershed. The area that contributes surface water runoff into a tributary system or water course.

Water Table. The surface of the groundwater below which the void spaces are completely saturated.

Waterway. (1) That portion of a watercourse which is actually occupied by water (2) A navigable inland body of water.

Wave. (1) An oscillatory movement of water on or near the surface of standing water in which a succession of crests and troughs advance while particles of water follow cyclic paths without advancing. (2) Motion of water in a flowing stream so as to develop the surficial appearance of a wave.

Wave Height. The vertical distance between a wave crest and the preceding trough.

Wave Length. The horizontal distance between similar points on two successive waves (e.g., crest to crest or trough to trough), measured in the direction of wave travel.

Wave Period. The time in which a wave crest travels a distance equal to one wave length. Can be measured as the time for two successive wave crests to pass a fixed point.

Weephole. A hole in a wall, invert, apron, lining, or other solid structure to relieve the pressure of groundwater.

Weir. A low overflow dam or sill for measuring, diverting, or checking flow.

Well. (1) Artificial excavation for withdrawal of water from underground storage. (2) Upward component of velocity in a stream.

Wetland. Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Windbreak. Barrier fence or trees to break or deflect the velocity of wind.

Windwave. A wave generated and propelled by wind blowing along the water surface.

Young. Immature, said of a stream on a steep gradient actively scouring its bed toward a more stable grade.

Topic 807 - Selected Drainage References

807.1 Introduction

Hydraulic and drainage related reference publications listed are grouped as to source.

807.2 Federal Highway Administration Hydraulic Publications

Copies of publications identified with an NTIS or GPO number may be ordered as follows:

NTIS - Send a check to:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

GPO - Send a check to:

Superintendent of Documents
Government Printing Office
Washington, D.C. 20402
(202) 783-3238

(1) Hydraulic Engineering Circulars (HEC).

HEC No.	Title	Date	FHWA # NTIS #
9	Debris-Control Structures	1971	EPD-86-106 PB86-179801/AS
11	Design of Riprap Revetment	2000	IF-00-022
14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2000	EPD-86-110 PB86-180205/AS
15	Design of Roadside Channels with Flexible Linings	2000	IF-00-022
17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	EPD-86-112 PB86-182110/AS
18	Evaluating Scour at Bridges	2001	NHI-01-001
20	Stream Stability at Highway Structures	2001	NHI-01-002
21	Bridge Deck Drainage Systems	1993	SA-92-010 PB94-109584
22	Urban Drainage Design Manual	1996	NHI-01-021
23	Bridge Scour and Stream Instability Countermeasures	2001	NHI-01-003
24	Highway Stormwater Pump Station Design	2001	NHI-01-007
25	Tidal Hydrology, Hydraulics, and Scour at Bridges	2004	NHI-05-077

(2) Hydraulic Design Series (HDS).

HDS No.	Title	Date	FHWA # NTIS #
1	Hydraulics of Bridge Waterways	1978	EPD-86-101 PB86-181708/AS
2	Highway Hydrology	2002	NHI-02-001
3	Design Charts for Open-Channel Flow	1961	EPD-86-102 PB86-179249/AS
4	Introduction to Highway Hydraulics	1997	HI-97-028 PB97-186761
5	Hydraulic Design of Highway Culverts (GPO 050-001-00298-1)	2004	IP-85-15 PB86-196961/AS
6	River Engineering for Highway Encroachments	2001	NHI-01-004

(3) Implementation Publications.

Title	Date	FHWA # NTIS #
Underground Disposal of Storm Water Runoff, Design Guidelines Manual	1980	TS-80-218 PB83-180257
Structural Design Manual for Improved Inlets and Culverts	1983	IP-83-6 PB84-153485
Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains	1984	TS-84-204 PB84-242585
Culvert Inspection Manual	1986	IP-86-2 PB87-151809

(4) Publications on CD-ROM.

Title	Date	FHWA # NTIS #
HDS-5 Hydraulic Design of Highway Culverts	(CDROM) v 1.00 1996	SA-96-080
Installation and User's Guide	1996	SA-96-081

(5) HYDRAIN - Integrated Drainage Design Computer System

All six volumes listed below are contained in report No. FHWA-SA-96-064.

Volume No.	Title
I	HYDRAIN - System Shell
II	HYDRO - Hydrology
III	HYDRA Storm Drains
IV	WSPRO - Step Backwater & Bridge Hydraulics
V	HY8 - Culvert Analysis
VI	HYCHL - Roadside Channels

807.3 American Association of State Highway and Transportation Officials (AASHTO)

(1) *Highway Drainage Guidelines*

The Drainage Guidelines is a collection of the guides previously published as individual volumes. These are:

I - Hydraulic Considerations in Highway Planning and Location

II - Hydrology

III - Erosion and Sediment Control in Highway Construction

IV - Hydraulic Design of Culverts

V - The Legal Aspects of Highway Drainage

VI - Hydraulic Analysis and Design of Open Channels

VII - Hydraulic Analysis for the Location and Design of Bridges

VIII- Hydraulic Aspects in Restoration and Upgrading of Highways

IX - Storm Drain Systems

X - Evaluating Highway Effects on Surface Water Environments

XI - Highways along Coastal Zones and Lakeshores

XII - Stormwater Management

XIII- Hydraulics Engineer Training and Career Development

XIV- Culvert Inspection and Rehabilitation

The current edition may be purchased through AASHTO, 444 North Capitol St., N.W., Suite 225, Washington D.C. 20001.

(2) *AASHTO Model Drainage Manual*

The Model Drainage Manual (MDM) is a comprehensive document covering a wide variety of transportation related hydraulic design issues. Developed for use by Federal, State, and local agencies, the MDM is a practice oriented document that allows the user agency to adopt the recommended values shown in the manual, or insert their own specific design policies and procedures.

807.4 California Department of Transportation

The following publications are available from the Caltrans Publications Unit, 1900 Royal Oaks Dr., Sacramento, CA 95815. Information on ordering and price can be checked by calling (916) 445-3520.

- Bridge Design Practice Manual
- Manual of Test - Volumes 1, 2, and 3
- Standard Plans
- Standard Specifications

807.5 U.S. Department of Interior - Geological Survey (USGS)

- Magnitude and Frequency of Floods in California - Water Resources Investigation 77-21.
- Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States - Open - File Report 93-419
- Guide For Determining Flood Flow Frequency - Bulletin #17B
- Water Resources Data for California, Part 1, Volumes 1 and 2.
- Rock Riprap Design for Protection of Stream Channels Near Highway Structures (1987) Volumes 1 and 2 (1987).

807.6 U.S. Department of Agriculture - Natural Resources Conservation Service (NRCS)

- Engineering Design Standards.
- Urban Hydrology for Small Watersheds - Technical Release 55

807.7 California Department of Water Resources and Caltrans

- Rainfall Intensity - Duration - Frequency Computer Program (Available through Caltrans).

807.8 University of California - Institute of Transportation and Traffic Engineering (ITTE)

- Street and Highway Drainage - Course Notes, Volumes 1 and 2.

807.9 U.S. Army Corps of Engineers

Publications and computer programs, too numerous to list, are available from the Water Resources Support Center. A publication catalog may be obtained by contacting the Hydrologic Engineering Center of the Corp, 609 Second St., Davis, CA 95616. The U. S. Army Corps of Engineers publications website address is: http://www.usace.army.mil/inet/usace-docs/.

Topic 808 – Selected Computer Programs

Table 808.1 below presents a software vs. capabilities matrix for hydrologic/hydraulic software packages that are approved for use by the Department. Where Caltrans drainage facilities connect or impact facilities that are owned by others, the affected Local Agency may require the Department to use a specific program that is not listed below. When the use of other computer programs is requested, a comparison with the results using the appropriate program from Table 808.1 should be made. However, when work is performed on projects under Caltrans' jurisdiction, either internally, or by others, if a program not listed in Table 808.1 is used, it should be demonstrated that the computations are based on the same principles that are used in the programs listed in Table 808.1. For information on Local Agency hydraulic computer program requirements, the District Hydraulics Branch should be contacted. It is the responsibility of the user to ensure that the version of the program being used from Table 808.1 is current.

Table 808.1

Summary of Related Computer Programs

	Storm Drains	Hydrology	Water Surface Profiles	Culverts	Roadside /Median Channels	Pavement Drainage	Pond Routing
HY-22					x	x	
TR-55		x					
HEC-HMS (2)		x					x
HY-8				x			
HEC-RAS (1)			x				
FESWMS			x				
HDS No 5: CD				x			
WMS		x		x			x
Caltrans IDF		x					
Hydraflow Storm Sewers	x					x	
Hydraflow Hydrographs		x					x

NOTES:

(1) The data that was used by FEMA to establish water surface elevations (usually HEC-2) must be used to develop a duplicate effective model for FEMA floodplain analysis. For more information contact FEMA or the Local Agency.

(2) HEC-1 has been superseded by HEC-HMS by the U.S. Army Corps of Engineers.

Special circumstances may dictate the use of alternative methods/programs. Any such use should be performed under direction and with approval of the District Hydraulics Engineer.

The use of flow length alone as a limiting factor for the Kinematic wave equation can lead to circumstances where the underlying assumptions are no longer valid. Over prediction of travel time can occur for conditions with significant amounts of depression storage, where there is high Manning's n-values or for flat slopes. One study suggests that the upper limit of applicability of the Kinematic wave equation is a function of flow length, slope and Manning's roughness coefficient. This study used both field and laboratory data to propose an upper limit of 100 for the composite parameter of $nL/s^{1/2}$. It is recommended that this criteria be used as a check where the designer has uncertainty on the maximum flow length to which the Kinematic wave equation can be applied to project conditions.

Where sheet flow travel distance cannot be determined, a conservative alternative is to assume shallow concentrated flow conditions without an independent sheet flow travel time conditions. See Index 816.6(2).

Table 816.6A
Roughness Coefficients For Sheet Flow

Surface Description	*n*
Hot Mix Asphalt	0.011-0.016
Concrete	0.012-0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
Woods(1)	
Light underbrush	0.40
Dense underbrush	0.80

(1) Woods cover is considered up to a height of 1 inch, which is the maximum depth obstructing sheet flow.

(2) Shallow concentrated flow travel time. After short distances, sheet flow tends to concentrate in rills and gullies, or the depth exceeds the range where use of the Kinematic wave equation applies. At that point the flow becomes defined as shallow concentrated flow. The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow. This method may also be used to calculate the total travel time for both the sheet flow and the shallow concentrated flow segments under certain conditions (e.g., where use of the Kinematic wave equation to predict sheet flow travel time is questionable, or where the designer cannot reasonably identify the point where sheet flow transitions to shallow concentrated flow).

Average velocities for the Upland Method can be taken directly from Figure 816.6 or may be calculated from the following equation:

$$V = (3.28)\ kS^{1/2}$$

Where S is the slope in percent and k is an intercept coefficient depending on land cover as shown in Table 816.6B.

Table 816.6B
Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	k
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled-alluvial fans	0.305
Grassed waterway	0.457

The travel time can be calculated from:

$$T_t = \frac{L}{60\ V}$$

where T_t is the travel time in minutes, L the length in feet, and V the flow velocity in feet per second.

(3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning equation, assuming bankfull conditions. See Index 864.3, Open Channel Flow Equations for further discussion of Manning's equation.

Appropriate values for "n", the coefficient of roughness in the Manning equation, may be found in most hydrology or hydraulics text and reference books. Table 864.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

Culvert or Storm Drain Flow. Flow velocities in a short culvert are generally higher than they would be in the same length of natural channel and comparable to those in a lined channel. In most cases, including short runs of culvert in the channel, flow time calculation will not materially affect the overall time of concentration (T_c). When it is appropriate to separate flow time calculations, such as for urban storm drains, Manning's equation may be used to obtain flow velocities within pipes.

The TR-55 library of equations for sheet flow, shallow concentrated flow and open channel flow is incorporated into the Watershed Modeling System (WMS) for Time of Concentration Calculations using Triangulated Irregular Networks (TINs) and Digital Elevation Maps (DEMs).

Topic 817 - Flood Magnitude

817.1 General

The determination of flood magnitude from either measurements made during a flood or after peak flow has subsided requires knowledge of open-channel hydraulics and flood water behavior. There are USGS Publications and other technical references available which outline the procedures for measuring flood flow. However, it is only through experience that accurate measurements can be obtained and/or correctly interpreted.

817.2 Measurements

(1) *Direct.* Direct flood flow measurements are those made during flood stage. The area and average velocity can be approximated and the estimated discharge can be calculated, from measurements of flow depth and velocity made simultaneously at a number of points in a cross section.

Discharges calculated from continuous records of stage gaging stations are the primary basis for estimating the recurrence interval or frequency of floods.

(2) *Indirect.* Indirect flood flow measurements are those made after the flood subsides. From channel geometry measurements and high water marks the magnitude of a flood can be calculated using basic open channel hydraulic equations given in Chapter 860. This method of determining flood discharges for given events is a valuable tool to the highway engineer possessing a thorough knowledge and understanding of the techniques involved.

Topic 818 - Flood Probability And Frequency

818.1 General

The estimation of peak discharges of various recurrence intervals is the most common and important problem encountered in highway engineering hydrology. Since the hydrology for the sizing of highway drainage facilities is concerned with future events, the time and magnitude of which cannot be precisely forecast, the highway engineer must resort to probability statistics to define the design discharge.

overtaxed. The alternative of accommodating the worst possible event that could happen is usually so costly that it may not be justified.

Highway engineers should understand that the option to select a predetermined design flood frequency is generally only applicable to new highway locations. Because of existing constraints, the freedom to select a prescribed design flood frequency may not exist for projects involving replacement of existing facilities. Caltrans policy relative to up-grading of existing drainage facilities may be found in Index 803.3.

Although the procedures and methodology presented in HEC 17, Design of Encroachments on Flood Plains Using Risk Analysis, are not fully endorsed by Caltrans, the circular is an available source of information on the theory of "least total expected cost (LTEC) design". Highway engineers are cautioned about applying LTEC methodology and procedures to ordinary drainage design problems. The Headquarters Hydraulics Engineer in the Division of Design should be consulted before committing to design by the LTEC method since its use can only be justified and recommended under extra-ordinary circumstances.

Topic 819 - Estimating Design Discharge

819.1 Introduction

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The estimation of peak discharge for various recurrence intervals is therefore the most important, and often the most difficult, task facing the highway engineer. Refer to Table 819.5A for a summary of methods for estimating design discharge.

In Topic 819, various design recommendations are given for both general and region-specific areas of California.

819.2 Empirical Methods

Because the movement of water is so complex, numerous empirical methods have been used in hydrology. Empirical methods in hydrology have great usefulness to the highway engineer. When correctly applied by engineers knowledgeable in the method being used and its idiosyncrasies, peak discharge estimates can be obtained which are functionally acceptable for the design of highway drainage structures and other features. Some of the more commonly used empirical methods for estimating runoff are as follows.

(1) *Rational Methods.* Undoubtedly, the most popular and most often misused empirical hydrology method is the Rational Formula:

$$Q = CiA$$

Q = Design discharge in cubic feet per second.

C = Coefficient of runoff.

i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration.

A = Drainage area in acres.

Rational methods are simple to use, and it is this simplicity that has made them so popular among highway drainage design engineers. Design discharge, as computed by these methods, has the same probability of occurrence (design frequency) as the frequency of the rainfall used. Refer to Topic 818 for further information on flood probability and frequency of recurrence.

An assumption that limits applicability is that the rainfall is of equal intensity over the entire watershed. Because of this, Rational Methods should be used only for estimating runoff from small simple watershed areas, preferably no larger than 320 acres. Even where the watershed area is relatively small but complicated by a mainstream fed by one or more significant tributaries, Rational Methods should be applied separately to each tributary stream and the tributary flows then routed down the main channel. Flow routing can best be accomplished through the use of hydrographs discussed under Index 816.5. Since Rational Methods give results that are in terms of instantaneous peak discharge and provide little information relative to runoff rate with respect to time, synthetic hydrographs should be developed for routing significant tributary inflows. Several relatively simple methods have been established for developing hydrographs, such as transposing a hydrograph from another hydrologically homogeneous watershed. The stream hydraulic method, and upland method are described in HDS No. 2. These, and other methods, are adequate for use with Rational Methods for estimating peak discharge and will provide results that are acceptable to form the basis for design of highway drainage facilities.

It is clearly evident upon examination of the assumptions and parameters which form the basis of the equation that much care and judgment must be applied with the use of Rational Methods to obtain reasonable results.

- The runoff coefficient "C" in the equation represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage.

Values of "C" may be determined for un-developed areas from Figure 819.2A by considering the four characteristics of: relief, soil infiltration, vegetal cover, and surface storage.

Some typical values of "C" for developed areas are given in Table 819.2B. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as:

$$C = \frac{C_1 A_2 + C_2 A_2 + ...}{A_1 + A_2 + ...}$$

- To properly satisfy the assumption that the entire drainage area contributes to the flow; the rainfall intensity, (i) in the equation expressed in inches per hour, requires that the storm duration and the time of concentration (t_c) be equal. Therefore, the first step in estimating (i) is to estimate (t_c). Methods for determining time of concentration are discussed under Index 816.6.

- Once the time of concentration, (t_c), is estimated, the rainfall intensity, (i), corresponding to a storm of equal duration, may be obtained from available sources such as intensity-duration-frequency (IDF) curves. See Index 815.3(3) for further information on IDF curves.

The runoff coefficients given in Figure 819.2A and Table 819.2B are applicable for storms of up to 5 or 10 year frequencies. Less frequent, higher intensity storms usually require modification of the coefficient because infiltration, detention, and other losses have a proportionally smaller effect on the total runoff volume. The adjustment of the rational method for use with major storms can be made by multiplying the coefficient by a frequency factor, C(f). Values of C(f) are given below. Under no circumstances should the product of C(f) times C exceed 1.0.

Frequency (yrs)	C(f)
25	1.1
50	1.2
100	1.25

(2) *Regional Analysis Methods.* Regional analysis methods utilize records for streams or drainage areas in the vicinity of the stream under consideration which would have similar characteristics to develop peak discharge estimates. These methods provide techniques for estimating annual peak stream discharge at any site, gaged or ungaged, for probability of recurrence from 50 percent (2 years) to 1 percent (100 years). Application of these methods is convenient, but the procedure is subject to some limitations.

Regional Flood - Frequency equations developed by the U.S. Geological Survey for use in California are given in Figure 819.2C and Table 819.7A. These equations are based on regional regression analysis of data from stream gauging stations. The equations in Figure 819.2C were derived from data gathered and analyzed through the mid-1970's, while the regions covered by Table 819.7A are reflective of a more recent (1994) study of the Southwestern U.S, which has been supplemented by a 2007 Study of California Desert Region Hydrology. Nomographs and complete information on use and development of this method may be found in "Magnitude and

Figure 819.2A

Runoff Coefficients for Undeveloped Areas

Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface storage, high; drainage system not sharply defined; large flood plain storage or large number of ponds or marshes

Given An undeveloped watershed consisting of;

1) rolling terrain with average slopes of 5%,
2) clay type soils,
3) good grassland area, and
4) normal surface depressions.

Find The runoff coefficient, C, for the above watershed.

Solution:

Relief	0.14
Soil Infiltration	0.08
Vegetal Cover	0.04
Surface Storage	0.06
C=	0.32

Table 819.2B

Runoff Coefficients for Developed Areas

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.25
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

Frequency of Floods in California" published in June, 1977 by the U.S. Department of the Interior, Geological Survey.

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. For example, the equations are not generally applicable to small basins on the floor of the Sacramento and San Joaquin Valleys as the annual peak data which are the basis for the regression analysis were obtained principally in the adjacent mountain and foothill areas. Likewise, the equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for urbanization. Further limitations on the use of USGS Regional Flood-Frequency equations are:

Region	Drainage Area (A) mi^2	Mean Annual Precip (P) in	Altitude Index (H) 1000 ft
(1)North Coast	0.2-3000	19-104	0.2-5.7
Northeast	0.2-25	all	all
Sierra	0.2-9000	7-85	0.1-9.7
Central Coast	0.2-4000	8-52	0.1-2.4
South Coast	0.2-600	7-40	all
(2)South Lahontan-Colorado Desert	0.2-90	all	all

Notes:
(1) In the North Coast region use a minimum value of 1 for altitude index (H)
(2) Use upper limit of 25 square miles

A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

(3) *Flood Frequency Analysis*

(a) If there are two gaged sites with similar watershed characteristics but one has a short record and the other has a longer record of peak flows, a two-station comparison analysis can be conducted to extend the equivalent length of record at the shorter gaged site.

(b) Flood-frequency relations at sites near gaged sites on the same stream (or in a similar watershed) can be estimated using a ratio of drainage area for the ungaged and gaged sites.

(c) At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional regression equations are considered the best estimates of flood frequency and are used to reduce the time-sampling error that may occur in a station flood-frequency estimate.

(d) The flood-frequency flows and the maximum peak discharges at several stations in a region should be used whenever possible for comparison with the peak discharge estimated at an ungaged site using a rainfall-runoff approach or regional regression equation. The watershed characteristics at the ungaged and gaged sites should be similar.

(4) *National Resources Conservation Service (NRCS) Methods.* The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 3 square miles (approx. 2,000 acres) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

Statistical methods of predicting stream discharge utilize numerical data to describe the process. Statistical methods, in general, do not require as much subjective judgment to apply as the previously described deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The accuracy of statistical methods can also be measured quantitatively. However, to assure that statistical method results are valid, the method and procedures used should be verified by an experienced engineer with a thorough knowledge of engineering statistics.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of recurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council Bulletin 17B, 1981, suggests at least 10 years of record are necessary to arrant astatistical analysis. The techniques of inferential statistics, the branch of statistics dealing with the inference of population characteristics, are described in HDS No. 2.

Before data on the specific characteristics to be examined can be properly analyzed, it must be arranged in a systematic manner. Several computer programs are available which may be used to systematically arrange data and perform the statistical computations.

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

Several standard frequency distributions have been studied extensively in the statistical analysis of hydrologic data. Those which have been found to be most useful are:

Figure 819.2C
Regional Flood-Frequency Equations

NORTH COAST REGION[2]

$Q_2 = 3.52\ A^{0.90}\ p^{0.89}\ H^{-0.87}$
$Q_5 = 5.04\ A^{0.89}\ p^{0.91}\ H^{-0.35}$
$Q_{10} = 6.21\ A^{0.88}\ p^{0.93}\ H^{-0.27}$
$Q_{25} = 7.64\ A^{0.87}\ p^{0.94}\ H^{-0.17}$
$Q_{50} = 8.57\ A^{0.87}\ p^{0.96}\ H^{-0.08}$
$Q_{100} = 9.23\ A^{0.87}\ p^{0.97}$

NORTHEAST REGION[3,4]

$Q_2 = 22\ A^{0.40}$
$Q_5 = 46\ A^{0.45}$
$Q_{10} = 61\ A^{0.49}$
$Q_{25} = 84\ A^{0.54}$
$Q_{50} = 103\ A^{0.57}$
$Q_{100} = 125\ A^{0.59}$

SOUTH LAHONTAN-COLORADO DESERT REGION[3,4]

$Q_2 = 7.3\ A^{0.30}$
$Q_5 = 53.0\ A^{0.44}$
$Q_{10} = 150\ A^{0.53}$
$Q_{25} = 410.0\ A^{0.63}$
$Q_{50} = 700.0\ A^{0.68}$
$Q_{100} = 1080.0\ A^{0.71}$

SIERRA REGION

$Q_2 = 0.24\ A^{0.88}\ p^{1.58}\ H^{-0.80}$
$Q_5 = 1.20\ A^{0.82}\ p^{1.37}\ H^{-0.64}$
$Q_{10} = 2.63\ A^{0.80}\ p^{1.25}\ H^{-0.58}$
$Q_{25} = 6.55\ A^{0.79}\ p^{1.12}\ H^{-0.52}$
$Q_{50} = 10.4\ A^{0.78}\ p^{1.06}\ H^{-0.48}$
$Q_{100} = 15.7\ A^{0.77}\ p^{1.02}\ H^{-0.43}$

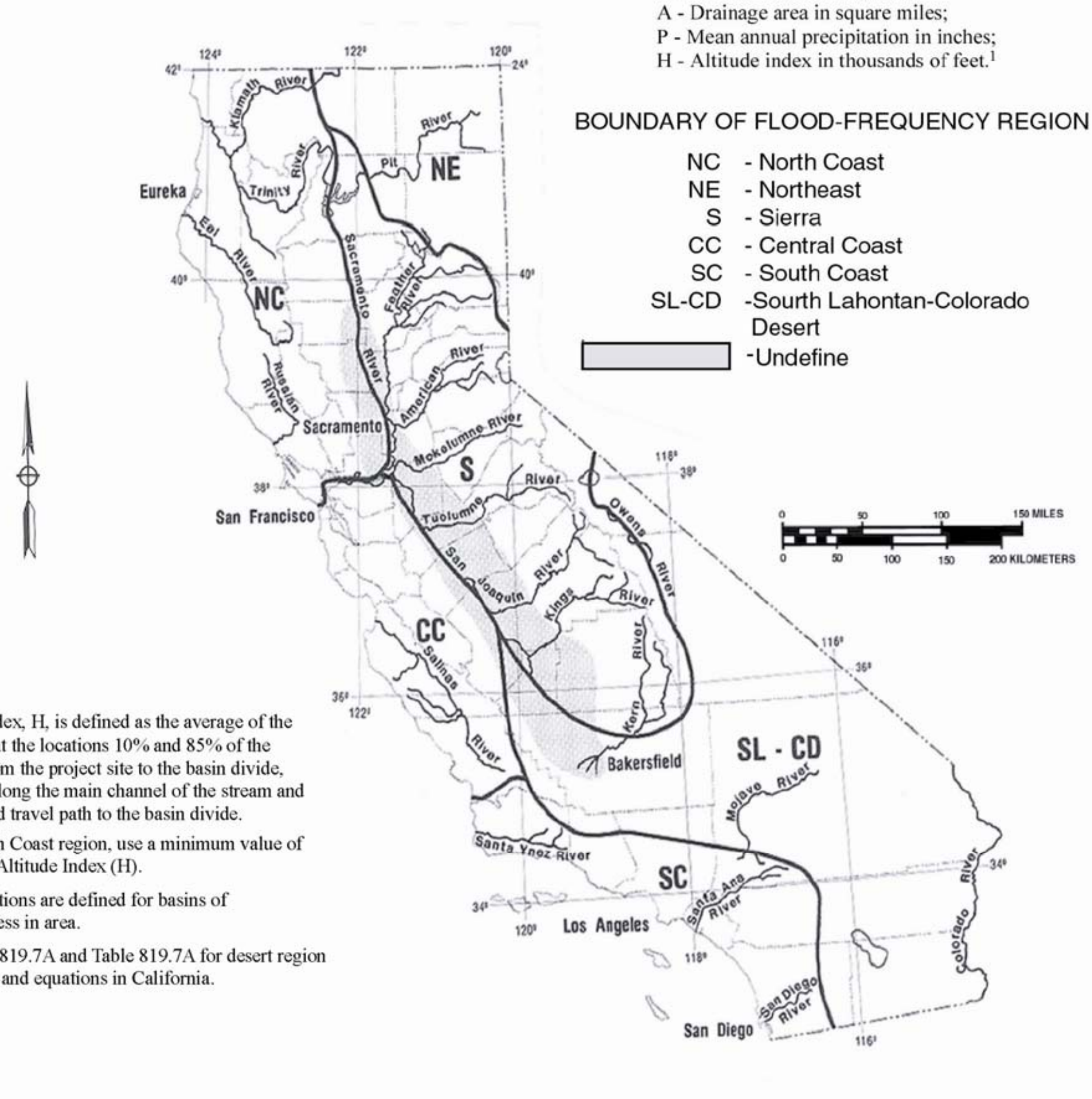
CENTRAL COAST REGION

$Q_2 = 0.0061\ A^{0.92}\ p^{2.54}\ H^{-1.10}$
$Q_5 = 0.118\ A^{0.91}\ p^{1.95}\ H^{-0.79}$
$Q_{10} = 0.583\ A^{0.90}\ p^{1.61}\ H^{-0.64}$
$Q_{25} = 2.91\ A^{0.89}\ p^{1.26}\ H^{-0.50}$
$Q_{50} = 8.20\ A^{0.89}\ p^{1.03}\ H^{-0.41}$
$Q_{100} = 19.7\ A^{0.88}\ p^{0.84}\ H^{-0.33}$

SOUTH COAST REGION

$Q_2 = 0.14\ A^{0.72}\ p^{1.62}$
$Q_5 = 0.40\ A^{0.77}\ p^{1.69}$
$Q_{10} = 0.63\ A^{0.79}\ p^{1.75}$
$Q_{25} = 1.10\ A^{0.81}\ p^{1.81}$
$Q_{50} = 1.50\ A^{0.82}\ p^{1.85}$
$Q_{100} = 1.95\ A^{0.83}\ p^{1.87}$

Q - Peak discharge in CFS, subscript indicates recurrence interval, in years;
A - Drainage area in square miles;
P - Mean annual precipitation in inches;
H - Altitude index in thousands of feet.[1]



NOTES:

1. Altitude Index, H, is defined as the average of the elevations at the locations 10% and 85% of the distance from the project site to the basin divide, measured along the main channel of the stream and the overland travel path to the basin divide.
2. In the North Coast region, use a minimum value of 1.0 for the Altitude Index (H).
3. These equations are defined for basins of 25 mi^2 or less in area.
4. See Figure 819.7A and Table 819.7A for desert region delineation and equations in California.

(1) *Log-Pearson Type III Distribution.* The popularity of the Log-Pearson III distribution is simply based on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. Because of this flexibility, the U.S. Water Resources Council recommends its use by all U.S. Government agencies as the standard distribution for flood frequency studies.

The three parameters necessary to describe the Log-Pearson III distribution are:

- Mean flow
- Standard deviation
- Coefficient of skew

Log-Pearson III distributions are usually plotted on log-normal probability graph paper for convenience even though the plotted frequency distribution may not be a straight line.

(2) *Log-normal Distribution.* The characteristics of the log-normal distribution are the same as those of the classical normal or Gaussian mathematical distribution except that the flood flow at a specified frequency is replaced with its logarithm and has a positive skew. Positive skew means that the distribution is skewed toward the high flows or extreme values.

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period of T_r = 2.33 years and that it has a positive skew.

Special probability paper has been developed for plotting log-normal and Gumbel distributions so that sample data, if it is distributed according to prescribed equations, will plot as a straight line.

819.4 Hydrograph Methods

Hydrograph methods of estimating design discharge relate runoff rates to time in response to a design storm. When storage must be considered, such as in reservoirs, natural lakes, and detention basins used for drainage or sediment control, the volume of runoff must be known. Since the hydrograph is a plot of flow rate against time, the area under the hydrograph represents volume. If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straight forward procedure. Rainfall records can be readily analyzed to estimate unit durations and the intensity which produces peak flows near the desired design discharge.

Hydrographs are also useful for determining the combined rates of flow for two drainage areas which peak at different times. Hydrographs can also be compounded and lagged to account for complex storms of different duration and varying intensities. Several methods of developing hydrographs are described in HDS No. 2. For basins without data, two of the most widely used methods described in HDS No. 2 for developing synthetic hydrographs are:

- Unit Hydrograph
- SCS Triangular Hydrograph

Both methods however tend to be somewhat inflexible since storm duration is determined by empirical relations.

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not available for a particular site but may exist at points upstream or in an adjacent or nearby watersheds.

(a) If the site is on the same stream and near a gaging station, peak discharges at the gaging station can be adjusted to the site by drainage area ratio and application of some appropriate power to each drainage area. The USGS may be helpful in suggesting appropriate powers to be used for a specific hydrologic region.

(b) If a design hydrograph can be developed at an upstream point in the same watershed, the procedure described in HDS No. 2 can be used to route the design hydrograph to the point of interest.

(c) If the site is somewhat removed from rain gage stations for which rainfall IDF curves have been computed, an interstation interpolation

Table 819.5A
Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	• Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration >or = concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage	Time of Concentration Drainage area Runoff coefficient Rainfall intensity
USGS Regional Regression Equations: USGS Water-Resources Investigation 77-21* Improved Highway Design Methods for Desert Storms	• Catchment area limit varies by region • Basin not located on floor of Sacramento or San Joaquin Valleys • Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel	Drainage area Mean annual precipitation Altitude index
NRCS (TR55)	• Small or midsize catchment (< 3 square miles) • Concentration time range from 0.1-10 hour (tabular hydrograph method limit < 2 hour) • Runoff is overland and channel flow • Simplified channel routing • Negligible channel storage	24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area
Unit Hydrograph (Gaged data) Synthetic Unit Hydrograph SCS Unit Hydrograph S-Graph Unit Hydrograph	• Midsize or large catchment (0.20 square miles to 1,000 square miles) • Uniformity of rainfall intensity and duration • Rainfall-runoff relationship is linear • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows	Rainfall hyetograph and direct runoff hydrograph for one or more storm events Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)
Statistical (gage data) Log-Pearson Type III Bulletin #17B – U.S. Department of the Interior	• Midsized and large catchments with stream gage data • Appropriate station and/or generalized skew coefficient relationship applied • Channel storage	10 or more years of gaged flood records
Basin Transfer of Gage Data	• Similar hydrologic characteristics • Channel storage	Discharge and area for gaged watershed Area for ungaged watershed

* Magnitude and Frequency of Floods in California

method is described in Volume I of DWR Bulletin No. 195 referenced in Index 815.3(3). Another method is by comparing the mean annual precipitation at the point of interest with that for nearby rain gage stations, the station most closely approximating the rainfall characteristics of the site can be selected.

819.6 Hydrologic Computer Programs

The rapid advancement of computer technology in recent years has resulted in the development of many mathematical models for the purpose of calculating runoff and other hydrologic phenomena. In the hands of knowledgeable and experienced engineers, good computer models are capable of efficiently calculating discharge estimates and other hydrologic results that are far more reliable than those which were obtained by other means. On the other hand, there is a tendency for the inexperienced engineer to accept computer generated output without questioning the reasonableness of the results obtained from a hydrologic viewpoint. Most computer simulation models require a significant amount of input data that must be carefully examined by a competent and experienced user to assure reliable results.

Some hydrologic computer models merely solve empirical hand methods more quickly. Other models are theoretical and solve the entire runoff cycle using mathematical equations to represent each phase of the runoff cycle.

In most simulation models, the drainage area is divided into subareas with similar hydrologic characteristics. A design rainfall is synthesized for each subarea, abstractions removed, and an overland flow routine simulates the movement of surface water into channels. The channels of the watershed are linked together and the channel flow is routed through them to complete the basin's response to the design rainstorm. Simulation models require calibration of modeling parameters using measured historical events to increase their validity.

A summary of personal computer programs is listed in Table 808.1.

Watershed Modeling System (WMS) is a comprehensive environment for hydrologic analysis. It was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (WES).

WMS merges information obtained from terrain models and GIS with industry standard hydrologic analysis models such as HEC-1 and TR-55. HY-8 has also been incorporated for culvert design.

Terrain models can obtain geometric attributes such as area, slope and runoff distances. Many display options are provided to aid in modeling and understanding the drainage characteristics of terrain surfaces.

The distinguishing difference between WMS and other applications designed for setting up hydrologic models like HEC-1 and TR-55 is its unique ability to take advantage of digital terrain for hydrologic data development.

WMS uses three primary data sources for model development:

1. Geographic Information Systems (GIS) Data
2. Digital Elevation Models (DEMs) published by the U.S. Geological Survey (USGS) at both 1:24,000 and 1:250,000 for the entire U.S. (the 1:24,000 data coverage is not complete)
3. Triangulated Irregular Networks (TINs)

Two other hydrologic computer programs that are commonly used are the Army Corps of Engineers' HEC-HMS and the National Resources Conservation Service's TR-20 Method.

Another computer program is the Caltrans Rainfall Intensity-Duration-Frequency (IDF) PC Program, which incorporates the California Department of Water Resources (DWR) short duration precipitation data (See Index 815.3(3)). The program eliminates reading values from graphs and simplifies the interpolation between rain gauge stations.

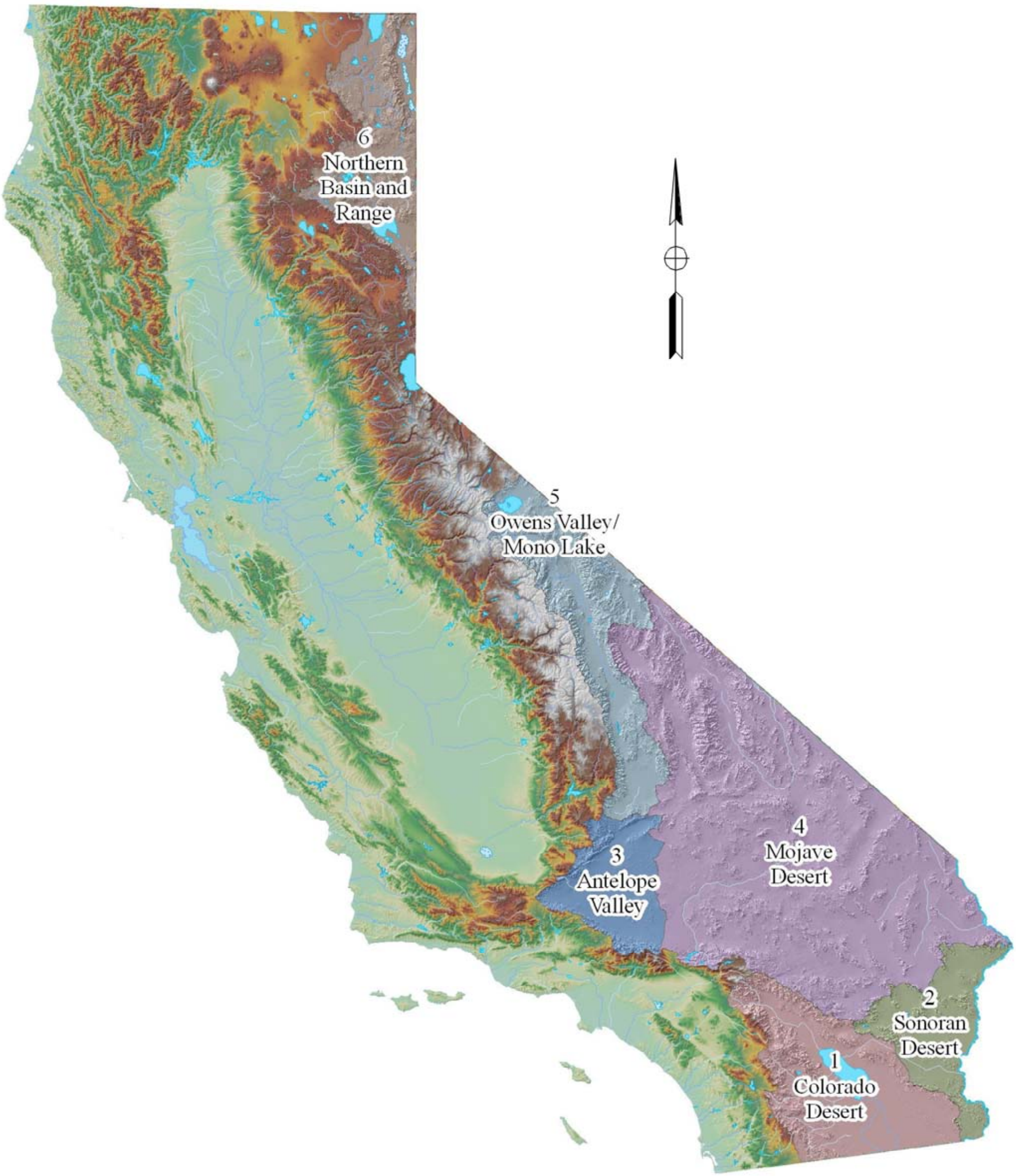
819.7 Region-Specific Analysis

(1) Desert Hydrology

Figure 819.7A shows the different desert regions in California, each with distinct hydrological characteristics that will be explained in this section.

Figure 819.7A

Desert Regions in California



(a) Storm Type

Summer Convective Storms - In the southern desert regions (Owens Valley/Mono Lake, Mojave Desert, Sonoran Desert and the Colorado Desert), the dominant storm type is the local thunderstorm, specifically summer convective storms. These storms are characterized by their short duration, over a relatively small area (generally less than 20 mi^2), and intense rainfall, which may result in flash floods. These summer convective storms may occur at any time during the year, but are most common and intense during the summer. General summer storms can also occur over these desert regions, but are rare, and usually occur from mid-August to early October. The rainfall intensity can vary from heavy rainfall to heavy thunderstorms.

General Winter Storm - In the Antelope Valley and Northern Basin and Range regions, the dominant storm type is the general winter storm. These storms are characterized by their long duration, 6 hours to 12 hours or more, and possibly intermittently for 3 days to 5 days over a relatively large area. General winter storms produce the majority of large peaks in the northern desert areas; the majority of the largest peaks discharge greater than or equal to 20 cfs/mi^2 occurred during the winter and fall months in the Owens Valley/Mono Lake and Northern Basin and Range regions. At elevations above 6,000 ft, much of the winter precipitation falls as snow; however, snowfall doesn't play a significant role in flood-producing runoff in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert). In the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range), more floods from snowmelt occur at lower elevations; more than 50 percent of runoff events occurred in spring, most likely snowmelt, but did not produce large floods.

(b) Regional Regression

Newly developed equations for California's Desert regions are shown on Table 819.7A.

While the regression equations for the Northern Basin and Range region provide more accurate results than previous USGS developed equations, there is some uncertainty associated with them. Therefore, the development of a rainfall-runoff model may be preferable for ungaged watersheds in this region.

(c) Rational Method

The recommended upper limit for California's desert regions is 160 acres (0.25 mi^2).

Table 819.7B lists common runoff coefficients for Desert Areas. These coefficients are applicable for storms with 2-year to 10-year return intervals, and must be adjusted for larger, less frequent storms by multiplying the coefficient by an appropriate frequency factor, C(f), as stated in Index 819.2(1) of this manual. The frequency factors, C(f), for 25-year, 50-year and 100-year storms are 1.1, 1.2 and 1.25, respectively. Under no circumstances should the product of C(f) times the runoff coefficient exceed 1.0. If a value of 1.0 is reached, it is recommended to use the value of 0.95.

(d) Rainfall-Runoff Simulation

A rainfall-runoff simulation approach uses a numerical model to simulate the rainfall-runoff process and generate discharge hydrographs. It has four main components: rainfall; rainfall losses; transformation of effective rainfall; and channel routing.

1. Rainfall

   a. Design Rainfall Criteria

      The selection of an appropriate storm duration depends on a number of factors, including the size of the watershed, the type of rainfall-runoff approach and hydrologic characteristics of the

Table 819.7A

Regional Regression Equations for California's Desert Regions

Region(s)	Associated Regression Equations
Colorado Desert Sonoran Desert Antelope Valley Mojave Desert	$Q_2 = 8.57A^{0.5668}$ $Q_5 = 80.32A^{0.541}$ $Q_{10} = 146.33A^{0.549}$ $Q_{25} = 291.04A^{0.5939}$ $Q_{50} = 397.82A^{0.6189}$ $Q_{100} = 557.31A^{0.6619}$
Owens Valley / Mono Lake	$Q_2 = 0.007A^{1.839}\left[\frac{ELEV}{1000}\right]^{1.485}\left[\frac{LAT-28}{10}\right]^{-0.680}$ $Q_5 = 0.212A^{1.404}\left[\frac{ELEV}{1000}\right]^{0.882}\left[\frac{LAT-28}{10}\right]^{-0.030}$ $Q_{10} = 1.28A^{1.190}\left[\frac{ELEV}{1000}\right]^{0.531}\left[\frac{LAT-28}{10}\right]^{0.525}$ $Q_{25} = 9.70A^{0.962}\left[\frac{ELEV}{1000}\right]^{0.107}\left[\frac{LAT-28}{10}\right]^{1.199}$ $Q_{50} = 34.5A^{0.829}\left[\frac{ELEV}{1000}\right]^{-0.170}\left[\frac{LAT-28}{10}\right]^{1.731}$ $Q_{100} = 111A^{0.707}\left[\frac{ELEV}{1000}\right]^{-0.429}\left[\frac{LAT-28}{10}\right]^{2.241}$

Table 819.7A

Regional Regression Equations for California's Desert Regions (Con't)

Northern Basin & Range	$Q_2 = 5.320A^{0.415}\left[\frac{H}{1000}\right]^{0.928}$ $Q_5 = 29.71A^{0.360}\left[\frac{H}{1000}\right]^{0.296}$ $Q_{10} = 85.76A^{0.314}\left[\frac{H}{1000}\right]^{-0.109}$ $Q_{25} = 275.5A^{0.253}\left[\frac{H}{1000}\right]^{-0.555}$ $Q_{50} = 616.9A^{0.281}\left[\frac{H}{1000}\right]^{-0.867}$ $Q_{100} = 1293A^{0.166}\left[\frac{H}{1000}\right]^{-1.154}$

Table 819.7B

Runoff Coefficients for Desert Areas

Type of Drainage Area	Runoff Coefficient
Undisturbed Natural Desert or Desert Landscaping (without impervious weed barrier)	0.30 – 0.40
Desert Landscaping (with impervious weed barrier)	0.55 – 0.85
Desert Hillslopes	0.40 – 0.55
Mountain Terrain (slopes greater than 10%)	0.60 – 0.80

Table 819.7C

Watershed Size for California Desert Regions

Desert Region	Duration (based on Watershed size)
Southern Regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert)	6-hour local storm (≤ 20 mi^2) 6-hour local storm and 24-hour general storm (between 20 mi^2 & 100 mi^2); use the larger peak discharge 24-hour general storm (> 100 mi^2)
Northern Regions (Owens Valley/Mono Lake and Northern Basin and Range)	24-hour general storm

study watershed. Watershed sizes are analyzed below and are applied to California's Desert regions in Table 819.7C.

Drainage Areas ≤ 20 mi^2 – Drainage areas less than 20 mi^2 are primarily representative of summer convective storms, and usually occur in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert regions). Since these storms usually result in intense rainfall, over a small drainage area and are generally less than 6 hours, it is recommended that a 6-hour local design storm be utilized.

Drainage Areas > 20 mi^2 & ≤ 100 mi^2 – For drainage areas between 20 mi^2 and 100 mi^2, the critical storm can be a summer convective storm or a general thunderstorm. For these drainage areas, it is recommended that both 6-hour and 24-hour design storm be analyzed, and the storm that produces the largest peak discharge be chosen as the design basis.

Drainage Areas > 100 mi^2 – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi^2, a 24-hour design storm is recommended.

b. Depth-Duration-Frequency Characteristics

In 2004, NOAA published updated precipitation-frequency estimates for arid regions of the southwestern United States, often cited as NOAA Atlas 14. This information is available online, via the Precipitation Frequency Data Server at http://hdsc.nws.noaa.gov/hdsc/pfds/ NOAA Atlas 14 supersedes NOAA's previous effort, NOAA Atlas 2, and California's

Department of Water Resources (DWR) Bulletin No. 195, where their coverages overlap.

NOAA Atlas 14 provides a vast amount of information, which includes:

- Point Estimates
- ESRI shapefiles and ArcInfo ASCII grids
- Color cartographic maps: all possible combination of frequencies (2-year to 1,000-year) and durations (5-miunte to 60-day)
- Associated Federal Geographic Data Committee-compliant metadata
- Data series used in the analysis: annual maximum series and partial duration series
- Temporal distributions of heavy precipitation (6-hour, 12-hour, 24-hour and 96-hour)
- Seasonal exceedance graphs: counts of events that exceed the 1 in 2, 5, 10, 25, 50 and 100 annual exceedance probabilities for the 60-minute, 24-hour, 48-hour and 10-day durations

In the areas where NOAA Atlas 14 data is not available, methodology from DWR's Bulletin No. 195 is recommended.

c. Depth-Area Reduction

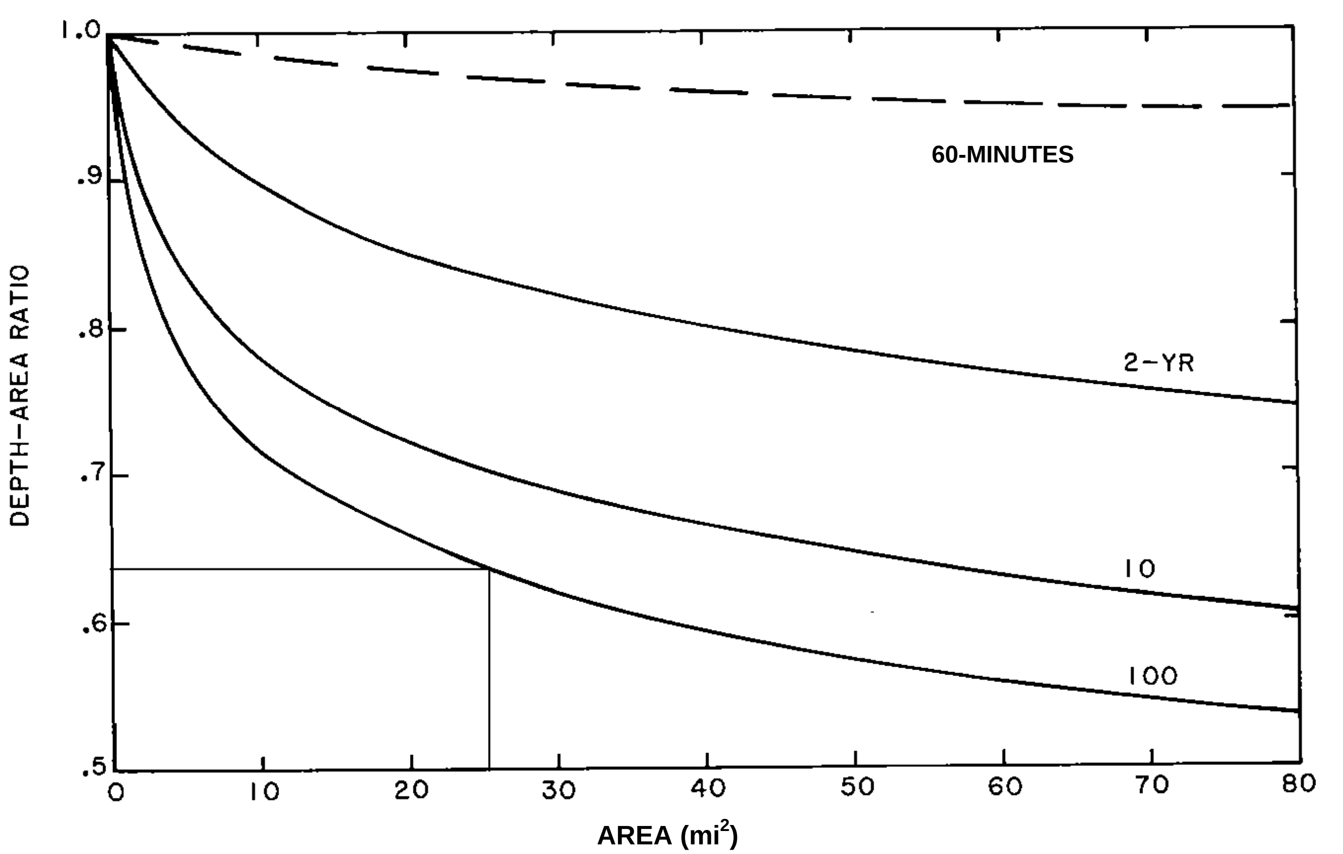
Depth-area reduction is the method of applying point rainfall data from one or several gaged stations within a watershed to that entire watershed. NOAA Atlas 14 provides high resolution depth-duration frequency point data which can then be computed with other depth-duration frequency data in that cell to obtain an average depth-duration frequency over a watershed. However, as this data is available as point data, the average calculated depth-duration frequency may not represent an entire watershed. To convert this point data into watershed area, a conversion factor may be applied, of which, two methods are available: applying a reduction factor; or applying depth-area reduction curves.

NOAA is currently working on updating the reduction factors, thus, until then, the depth-area reduction curves are recommended. Two depth-area reduction curves are available: (1) the depth curves in National Weather Service's HYDRO-40 (http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and the Mojave Desert). For the upper regions (Owens Valley/Mono Lake and Northern Basin and Range), the curves from NOAA Atlas 2 are recommended.

The variables needed to apply depth area reduction curves to a watershed are a storm frequency (i.e., a 100-year storm), storm duration (i.e., a 30-minutes storm), and the area of a watershed. For example, if a 100-year storm with a duration of 60-minutes were to be analyzed over a desert watershed of 25 mi^2, then using Figure 819.7B, the Depth-Area Ratio would be 0.64. This ratio would then be multiplied by the averaged point-rainfall data, which would then

Figure 819.7B

Example Depth-Area Reduction Curve



result in the rainfall over the entire watershed.

Point rainfall data is available from NOAA Atlas 14, which must then be converted to area rainfall data. Conversions are available in two forms: (1) the National Weather Service's HYDRO-40, and (2) NOAA Atlas 2. The National Weather Service's HYDRO-40 is recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert.) NOAA Atlas 2 is recommended for the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range).

2. Rainfall Losses

Antecedent Moisture Condition – The Antecedent Moisture Condition (AMC) is the amount of moisture present in the soil before a rainfall event, or conversely, the amount of moisture the soil can absorb before becoming saturated (Note: the AMC is also referred to as the Antecedent Runoff Condition [ARC]). Once the soil is saturated, runoff will occur. Generally, the AMC is classified into three levels:

- AMC I – Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.
- AMC II – Moderate runoff potential. AMC II represents an average study condition.
- AMC III – Highest runoff potential. The watershed is practically saturated from antecedent rainfall.

Because of the different storm types present in California's desert regions, AMC I is recommended as design criteria for local thunderstorms, and AMC II is recommended as design criteria for general storms.

Curve Number – The curve number was developed by the then Soil Conservation Service (SCS), which is now called the National Resource Conservation Service (NRCS). The curve number is a function of land use, soil type and the soil's AMC, and is used to describe a drainage area's storm water runoff potential. The soil type(s) are typically listed by name and can be obtained in the form of a soil survey from the local NRCS office. The soil surveys classify and present the soil types into 4 different hydrological groups, which are shown in Table 819.7D. From the hydrological groups, curve numbers are assigned for each possible land use-soil group combinations, as shown in Table 819.7E. The curve numbers shown in Table 819.7E are representative of AMC II, and need to be converted to represent AMC I, and AMC III, respectively. The following equations to convert an AMC II curve number to an AMC I or AMC III curve number, using a five-day period as the minimum for estimating the AMC's:

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

Note: The AMC of a storm area may vary during a storm; heavy rain falling on AMC I soil can change the AMC from I to II or III during the storm.

3. Transformation

Total runoff can be characterized by two types of runoff flow: direct runoff and base flow. Direct runoff is classified as storm runoff occurring during or shortly after a storm event. Base flow is classified as subsurface runoff from prior precipitation events and delayed subsurface runoff from the current storm. The transformation of precipitation runoff to excess can be

Table 819.7D

Hydrologic Soil Groups

Hydrologic Soil Group	Soil Group Characteristics
A	Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

accomplished using a unit hydrograph approach. The unit hydrograph method is based on the assumption that a watershed, in converting precipitation excess to runoff, acts as a linear, time-invariant system.

Unit Hydrograph Approach

A unit hydrograph for a drainage area is a curve showing the time distribution of runoff that would result at the concentration point from one inch of effective rainfall over the drainage area above that point.

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, that the time factors that affect the unit hydrograph shape are invariant, and that watershed rainfall-runoff relationships are characterized by watershed area, slope and shape factors.

a. SCS Unit Hydrogoraph

The SCS dimensionless unit hydrograph is based on averages of unit hydrographs derived from gaged rainfall and runoff for a large number of small rural basins throughout the U.S. The definition of the SCS unit hydrograph normally only requires one parameter, which is lag, defined as the time from the centroid of precipitation excess to the time of the peak of the unit hydrograph. For ungaged watersheds, the SCS suggests that the unit hydrograph lag time, t_{lag}, may be related to time of concentration t_c, through the following relation:

$$t_{lag} = 0.6t_c$$

The time of concentration is the sum of travel time through sheet flow, shallow concentrated flow, and channel flow segments. A typical SCS Unit Hydrograph is similar to Figure 816.5.

A unit hydrograph can be derived from observed rainfall and runoff, however either may be unavailable. In such cases, a synthetic unit hydrograph can be developed using the S-graph method.

Table 819.7E

Curve Numbers for Land Use-Soil Combinations

Description	Average % Impervious	Curve Number by Hydrological Soil Group				Typical Land Uses
		A	B	C	D	
Residential (High Density)	65	77	85	90	92	Multi-Family, Apartments, Condos, Trailer Parks
Residential (Medium Density)	30	57	72	81	86	Single-Family, Lot Size ¼ to 1 acre
Residential (Low Density)	15	48	66	78	83	Single-Family, Lot Size 1 acre or greater
Commercial	85	89	92	94	95	Strip Commercial, Shopping Centers, Convenience Stores
Industrial	72	81	88	91	93	Light Industrial, Schools, Prisons, Treatment Plants
Disturbed / Transitional	5	76	85	89	91	Gravel Parking, Quarries, Land Under Development
Agricultural	5	67	77	83	87	Cultivated Land, Row Crops, Broadcast Legumes
Open Land – Good	5	39	61	74	80	Parks, Golf Courses, Greenways, Grazed Pasture
Meadow	5	30	58	71	78	Hay Fields, Tall Grass, Ungrazed Pasture
Woods (Thick Cover)	5	30	55	70	77	Forest Litter and Brush adequately cover soil
Woods (Thin Cover)	5	43	65	76	82	Light Woods, Woods-Grass Combination, Tree Farms
Impervious	95	98	98	98	98	Paved Parking, Shopping Malls, Major Roadways
Water	100	100	100	100	100	Water Bodies, Lakes, Ponds, Wetlands

b. S-graph

An S-graph is a summation hydrograph of runoff that would result from the continuous generation of unit storm effective rainfall over the area (1-inch per hour continuously). The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungaged basins, and is based upon percent of ultimate discharge and percent of lag time. Several entities, including local and Federal agencies, have developed location-specific S-Graphs that areapplicable to California's desert regions.

The ordinate is expressed in percent of ultimate discharge, and the abscissa is expressed in percent of lag time. Ultimate discharge, which is the maximum discharge attainable for a given intensity, occurs when the rate of runoff on the summation hydrograph reaches the rate of effective rainfall.

Lag for a watershed is an empirical expression of the hydrologic characteristics of a watershed in terms of time. It is defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable ungaged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungaged watersheds can be easily derived.

Figure 819.7C is a sample illustration of a San Bernardino County S-Graph, while Figure 819.7D shows an example S-Graph from USBR.

Recommendations

For watersheds with mountainous terrain/high elevations in the upper portions, the San Bernardino County Mountain S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. For watersheds in the southern desert regions with limited or no mountainous terrain/high elevations, the San Bernardino County Desert S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. The U.S. Bureau of Reclamation (USBR) S-Graph (http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf) is recommended for watersheds in the Northern Basin and Range.

As an alternative to the above mentioned S-Graphs, the SCS Unit Hydrograph may also be used.

(4) Channel Routing

Channel routing is a process used to predict the temporal and spatial variation of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. The four commonly used methods are the kinematic wave routing, Modified Puls routing, Muskingum routing, and Muskingum-Cunge routing. The advantages and disadvantages for each method are described in Table 819.7F. Table 819.7G provides guidance for selecting an appropriate routing method

Figure 819.7C

San Bernardino County Hydrograph for Desert Areas

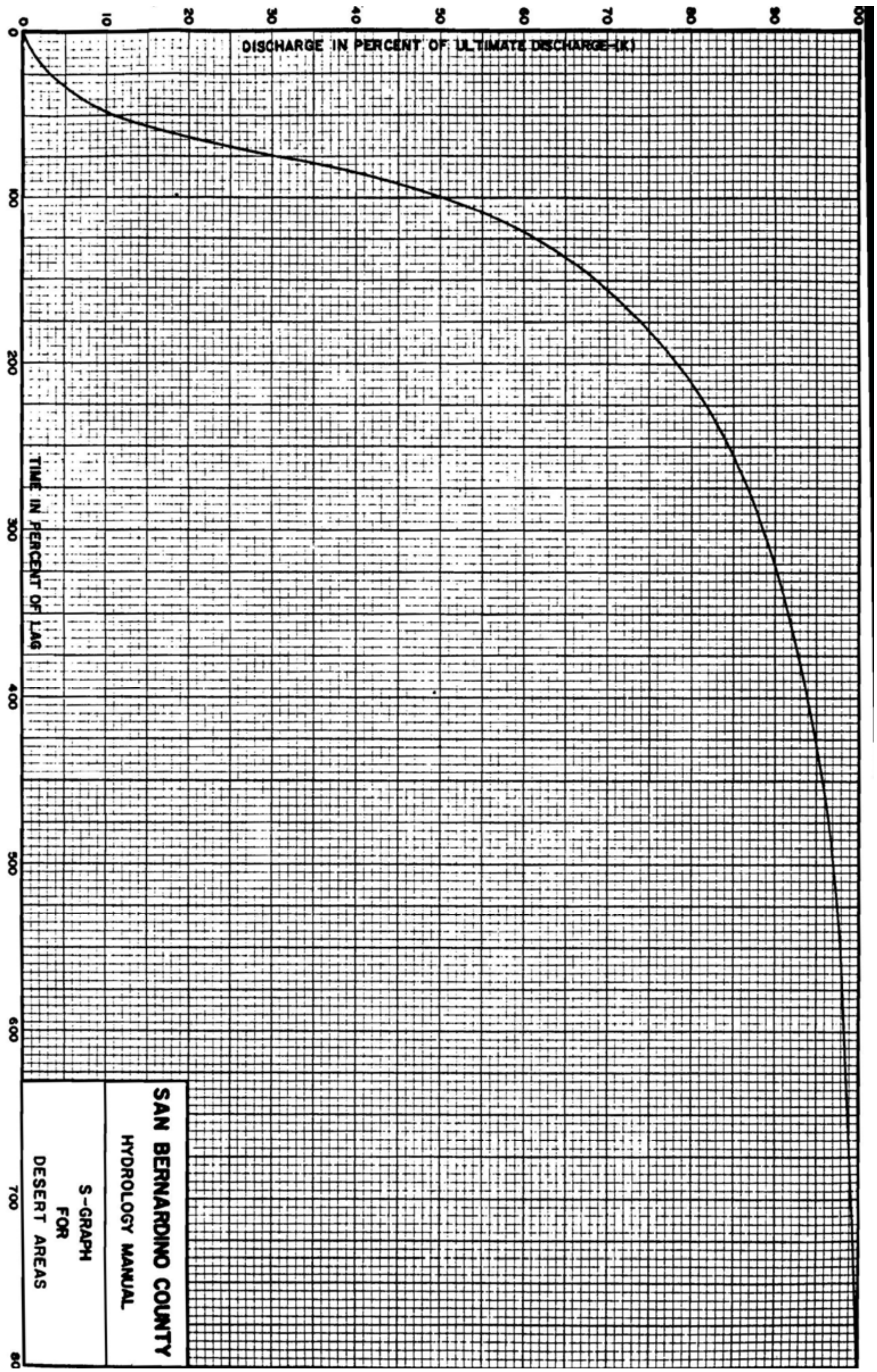


Figure 819.7D

USBR Example S-Graph

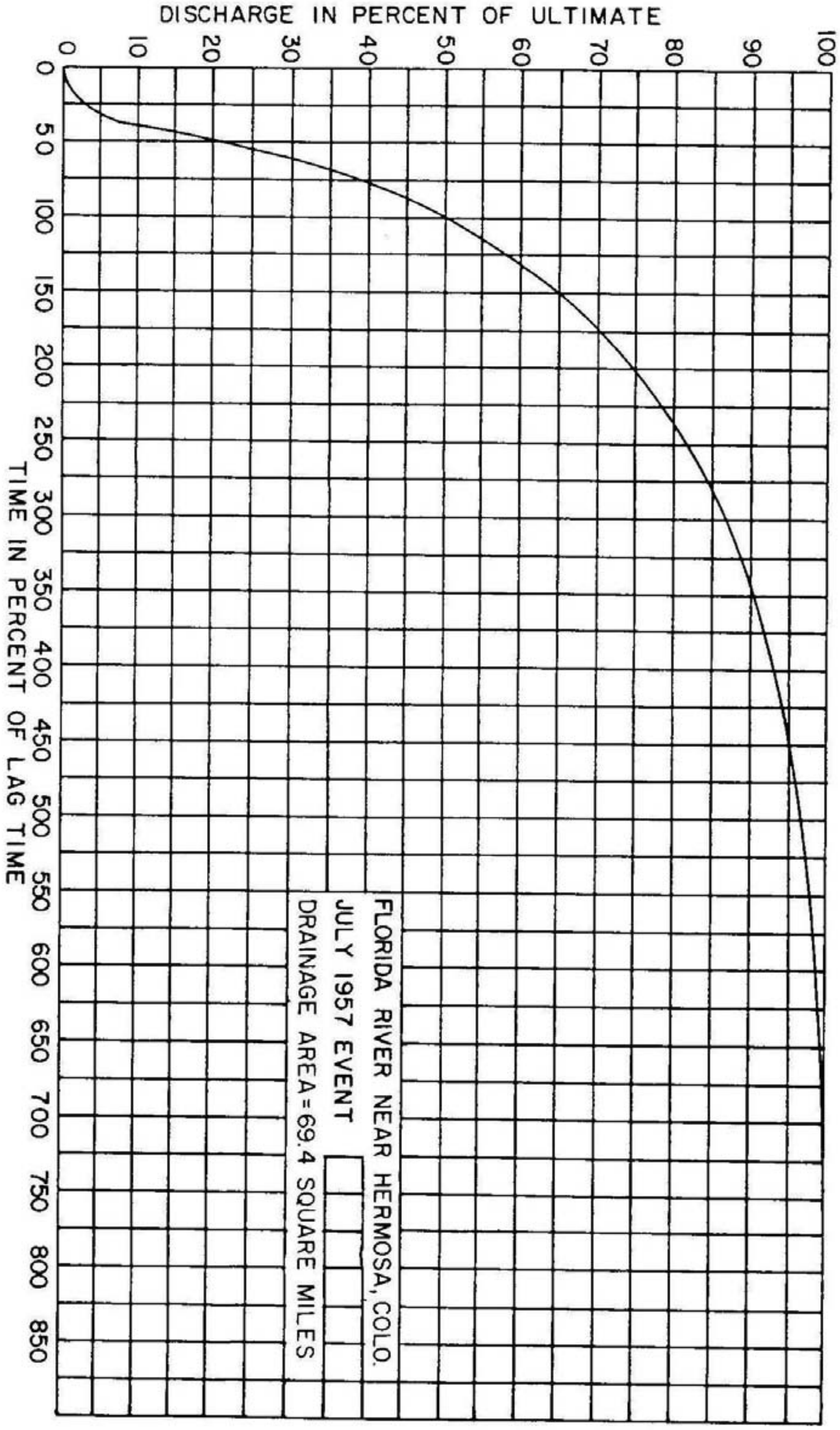


Table 819.7F

Channel Routing Methods

Routing Method	Pros	Cons
Kinetmatic Wave	▪ A conceptual model assuming a uniform flow condition. ▪ In general, works best for steep (10 ft/mile or greater), well defined channels. ▪ It is often applied in urban areas because the routing reaches are generally short and well-defined.	▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship.	▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion.	▪ The coefficients cannot be used to model a range of floods that may remain in bank or go out of bank. Therefore, not applicable to significant overbank flows.
Muskingum-Cunge	▪ A nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. ▪ The parameters are physically based. ▪ Has been shown to compare well against the full unsteady flow equations over a wide range of flow conditions.	▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

Table 819.7G

Channel Method Routing Guidance

If this is true…	… then this routing model may be considered.
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain.	Modified Puls; Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; Modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Notes:

T = hydrograph duration
u_o = reference mean velocity
d_o = reference flow depth
S_o = channel slope

The Muskingum-Cunge routing method can handle a wide range of flow conditions with the exception of significant backwater. The Modified Puls routing can model backwater effects. The kinematic wave routing method is often applied in urban areas with well defined channels.

(5) Storm Duration and Temporal Distribution

Temporal distribution is the time-related distribution of the precipitation depth within the duration of the design storm. Temporal distribution patterns of design storms are based on the storm duration. The temporal distribution pattern for short-duration storms represents a single cloudburst and is based on rainfall statistics. The temporal distribution for long-duration storms resembles multiple events and is patterned after historic events.

Since the storm events in California's desert regions are made up of two distinct separate storm types, the summer convective storm and the general winter storm, the design storm durations should be adjusted accordingly. For California's desert regions, the 100-year 6-hour storm is recommended for the convective storms, and the 100-year 24-hour storm is recommended for the winter storms. Table 819.7H summarizes the design storm durations for the different desert regions throughout California.

(2) Sediment/Debris Bulking

The process of increasing the water volume flow rate to account for high concentrations of sediment and debris is defined as bulking. Debris carried in the flow can be significant and greatly increase flow volume conveyed from a watershed. This condition occurs frequently in mountainous areas subject to wildfires with soil erosion, as well as arid regions around alluvial fans and other geologic activity. By bulking the flow through the use of an appropriate bulking factor, bridge openings and culverts can be properly sized for areas that experience high sediment and debris concentration.

(a) Bulking Factor

Bulking factors are applied to a peak (clear-water) flow to obtain a total or bulked peak flow, which provides a safety factor in the sizing of hydraulic structures. For a given watershed, a bulking factor is typically a function of the historical concentration of sediment in the flow.

(b) Types of Sediment/Water Flow

The behavior of flood flows will vary depending on the concentration of sediment in the mixed flow, where the common flow types are normal stream flow, hyperconcentrated flow, and debris flow.

1. Normal Stream Flow

During normal stream flow, the sediment load minimally influences flow behavior or characteristics. Because sediment has little impact, this type of flow can be analyzed as a Newtonian fluid and standard hydraulic methods can be used. The upper limit of sediment concentration by volume for normal stream flow is 20 percent and bulking factors are applied cautiously because of the low concentration. (See Table 819.7I) The small amount of sediment is conveyed by conventional suspended load and bed-load.

2. Hyperconcentrated Flow

Hyperconcentrated flow is more commonly known as mud flow. Because of potential for large volumes of sand in the water column, fluid properties and transport characteristics change and the mixture does not behave as a Newtonian fluid. However, basic hydraulic methods and models are still generally accepted and used for up to 40 percent sediment concentration by volume. For hyperconcentrated flow, bulking factors vary between 1.43 and 1.67 as shown in Table 819.7I.

3. Debris Flow

In debris flow state, behavior is primarily controlled by the composition of the sediment and debris mixture, where the volume of clay can have a strong influence in the yield strength of the mixture.

During debris flow, which has an upper limit of 50 percent sediment concentration by volume, the sediment/debris/water mixture no longer acts as a Newtonian fluid and basic hydraulic equations do not apply. If detailed hydraulic analysis or modeling of a stream operating under debris flow is needed, FLO2DH is the recommended software choice given its specific debris flow capabilities. HEC-RAS is appropriate for normal stream flow and hyperconcentrated flow, but cannot be applied to debris flow.

For a typical debris flow event, clear-water flow occurs first, followed by a frontal wave of mud and debris. Low frequency events, such as the 100-year flood, most likely contain too much water to produce a debris flow event. Normally, smaller higher frequency events such as 10-year or 25-year floods actually have a greater probability of yielding a debris flow event requiring a higher bulking factor.

As outlined in Table 819.7I, bulking factors for debris flow vary between 1.67 and 2.00.

(c) Sediment/Debris Flow Potential

1. Debris Hazard Areas

Mass movement of rock, debris, and soil is the main source of bulked flows. This can occur in the form of falls, slides, or flows. The volume of sediment and debris from mass movement can enter streams depending upon hydrologic and geologic conditions.

Table 819.7H

Design Storm Durations

Drainage Area	Desert Region	100-year, 6-hour Convective Storm (AMC I)	100-year, 24-hour General Storm (AMC II)	Regional Regression Equations
> 20 mi^2	Colorado Desert	X		
	Sonoran Desert	X		
	Mojave Desert	X		
	Antelope Valley Desert	X		
< 20 mi^2	Colorado Desert	X*	X*	
	Sonoran Desert	X*	X*	
	Mojave Desert	X*	X*	
	Antelope Valley Desert	X*	X*	
	Owens Valley/Mono Lake			X**
	Northern Basin & Range		X	

* For watersheds greater than 20 mi^2 in the southern desert regions, both the 6-hour Convective Storm (AMC I) and the 24-hour General Storm (AMC II) should be analyzed and the larger of the two peak discharges selected.

** The use of regional regression equations is recommended where streamgage data are not available; otherwise, hydrologic modeling could be performed with snowmelt simulation.

Table 819.7I

Bulking Factors & Types of Sediment Flow

Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight	Sediment Concentration by Volume
		(100% by WT = 1 x 10^6 ppm)	(specific gravity = 2.65)
Normal Streamflow	0	0	0
	1.11	23	10
	1.25	40	20
Hyperconcentrated Flow	1.43	52	30
	1.67	53	40
Debris Flow	2.00	72	50
Landslide	2.50	80	60
	3.33	87	70

The location of these debris-flow hazards include:

(1) At or near the toe of slope 2:1 or steeper

(2) At or near the intersection of ravines and canyons

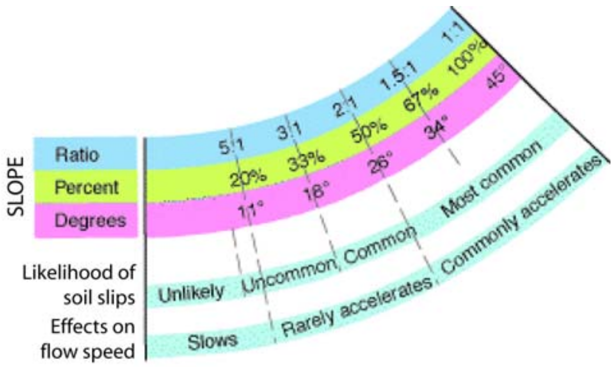
(3) Near or within alluvial fans

(4) Soil Slips

Soil slips commonly occur at toes of slope between 2:1 and 3:1. Flowing mud and rocks will accelerate down a slope until the flow path flattens. Once energy loss occurs, rock, mud, and vegetation will be deposited. Debris flow triggered by soil slips can become channelized and travel distances of a mile or more. Figure 819.7E shows the potential of soil slip versus slope angle. As seen in this Figure, the flatter the slope angle, the less effect on flow speed and acceleration.

Figure 819.7E

Soil Slips vs. Slope Angle



2. Geologic Conditions

In the Transverse Ranges that include the San Gabriel and San Bernardino Mountains along the southern and southwestern borders of the Antelope Valley (Region 3) and Mojave Desert (Region 4), their substrate contains sedimentary rocks, fractured basement rocks, and granitic rocks. This type of geology has a high potential of debris flow from the hillsides of these regions.

While debris flow potential is less prevalent, it is possible to have this condition in the Peninsula Ranges that include the San Jacinto, Santa Rosa, and Laguna Mountains along the western border of the Colorado Desert (Region 1).

(d) Alluvial Fans

An alluvial fan is a landform located at the mouth of a canyon, formed in the shape of a fan, and created over time by deposition of alluvium. With the apex of the fan at the mouth of a canyon, the base of the fan is spread across lower lying plains below the apex. Over time, alluvial fans change and evolve when sediment conveyed by flood flows or debris flows is deposited in active channels, which creates a new channel within the fan. Potentially, alluvial fan flood and debris flows travel at high velocity, where large volumes of sediment can be eroded from mountain canyons down to the lower fan surface. Given this situation, the alignments of the active channels and the overall footprint of an alluvial fan are dynamic. Also, the concentration of sediment/debris volume is dynamic, ranging from negligible to 50 percent.

Alluvial fans can be found on soil maps, geologic maps, topographic maps, and aerial photographs, in addition to the best source which is a site visit. An example of an alluvial fan, shown in plan view, is in Figure 819.7F and Figure 872.3.

(e) Wildfire and Debris Flow

After fires have impacted a watershed, sediment/debris flows are caused by surface erosion from rainfall runoff and landsliding due to rainfall infiltration into the soil. The most dominant cause is the runoff process because fire generally reduces the infiltration and storage capacity of soils, which increases runoff and erosion.

Figure 819.7F

Alluvial Fan

1. Fire Impacts

   Arid regions do not have the same density of trees and vegetation as a forested area, but the arid environment still falls victim to fires in a similar manner. Prior to a fire, the arid region floor can contain a litter layer (leaves, needles, fine twigs, etc.), as well as a duff layer (partially decomposed components of the litter layer). These layers absorb water, provide storage of rainfall, and protect hillsides. Once these layers are burned, they become ash and charcoal particles that seal soil pores and decrease infiltration potential of the soil, which ultimately increases runoff and erosion.

   In order to measure the burn severity of watersheds with respect to hydrologic function, classes of burn severity have been created. These classes are simply stated as high, moderate, low, and unburned. From moderate and high burn severity slopes, the generated sediment can reach channels and streams causing bulked water flows during storm events. Generally speaking, the denser the vegetation in a watershed prior to a fire and the longer a fire burns within this watershed, the greater the effects on soil hydrologic function. This occurs due to the fire creating a water repellent layer at or near the soil surface, the loss of soil structural stability, which all results in more runoff and erosion. After a one or two-year period, the water repellent layer is usually washed away.

(f) Local Agency Methods For Predicting Bulking Factors

1. San Bernardino County

   Instead of conducting a detailed analysis, San Bernardino Flood Control District uses a set value for bulking of 2 (i.e., 100 percent bulking) for any project where bulking flows may be anticipated. This bulking factor of 2 can also be expressed as a 50 percent sediment concentration by volume, which is about the upper limit of debris flow. A higher percentage of sediment concentration would be considered a landslide instead of debris flow. Basically, the San Bernardino County method assumes debris flow conditions for all types of potential bulking.

2. Los Angeles County

   The Los Angeles (LA) County method uses a watershed-specific bulking factor. The LA County Sedimentation Manual, which is located at http://ladpw.org/wrd/publication, divides the county into three basins: LA Basin, Santa Clara River Basin, and Antelope Valley, where only the latter is located in the Caltrans desert hydrology regions. The production of sediment from these basins is dependent upon many factors, including rainfall intensity, vegetative cover, and watershed slope. For each of the LA

County basins, Debris Potential Area (DPA) zones have been identified.

The Design Debris Event (DDE) is associated with the 50-year, 24-hour duration storm, and produces the quantity of sediment from a saturated watershed that is recovered from a burn. For example, a DPA 1 zone sediment rate of 120,000 cubic yards per square mile has been established as the DDE for a 1-square mile drainage area. This sediment rate is recommended for areas of high relief and granitic formation found in the San Gabriel Mountains. In other mountainous areas in LA County, lower sediment rates have been assigned based on differences in topography, geology, and precipitation. For the Antelope Valley basin, eight debris production curves have been generated, and can be found in Appendix B of the LA County Sedimentation Manual along with curves for the other basins.

In addition to sediment production rates, a series of peak bulking factor curves are presented for each LA County basin in Appendix B of the LA manual. The peak bulking factor can be estimated using these curves based on the watershed area and the DPA. Within the Antelope Valley basin, maximum peak bulking factors range from 1.2 in DPA Zone 11 to 2.00 in DPA Zone 1.

3. Riverside County

For Riverside County, a bulking factor is calculated by estimating a sediment/debris yield rate for a specific storm event, and relating it to the largest expected sediment yield of 120,000 cubic yards per square mile for a 1-square mile watershed from the LA County procedure. This sediment rate from LA County is based on the DPA Zone 1 corresponding to the highest expected bulking factor of 2.00.

The bulking factor equation from the Riverside County Hydrology Manual (http://www.floodcontrol.co.riverside.ca.us/downloads/planning/) is as follows:

$$BF = 1 + \frac{D}{120{,}000}$$

BF = Bulking Factor

D = Design Storm Sediment/Debris Production Rate For Study Watershed (cubic yards/square mile)

4. U.S. Army Corps of Engineers- LA District

This method, located at http://www.spl.usace.army.mil/resreg/htdocs/Publications.html, was originally developed to calculate unit sediment/debris yield values for an "n-year" flood event, and applied to the design and analysis of debris catching structures in coastal Southern California watersheds. The LA District method considers frequency of wildfires and flood magnitude in its calculation of unit debris yield. Even though its original application was intended for coastal-draining watersheds, this method can also be used for desert-draining watersheds for the same local mountain ranges.

The LA District method can be applied to watershed areas between 0.1 and 200 mi^2 that have a high proportion of their total area in steep, mountainous topography. This method is best used for watersheds that have received significant antecedent rainfall of at least 2 inches in 48 hours. Given this criteria, the LA District method is more suited for general storms rather than thunderstorms.

As shown below, this method specifies a few equations to estimate unit debris yield dependent upon the areal size of the watershed. These equations were developed by multiple regression analysis using known sediment/debris data.

For watersheds between 3 and 10 mi^2, the following equations can be used:

$$\log Dy = 0.85 \log Q + 0.53 \log RR$$
$$+0.04 \log A + 0.22FF$$

D_y = Unit Debris Yield (cubic yards/square mile)

RR = Relief Ratio (foot/mile), which is the difference in elevation between the highest and lowest points on the longest watercourse divided by the length of the longest watercourse

A = Drainage Area (acres)

FF = Fire Factor

Q = Unit Peak Runoff (cfs/square mile)

In order to account for increase in debris yield due to fire, a non-dimensional fire factor (FF) is a component in the equation above. The FF varies from 3.0 to 6.5, with a higher factor indicating a more recent fire and more debris yield. This factor is 3.0 for desert watersheds because the threat and effects from fire are minimal.

Because the data used to develop the regression equation was taken from the San Gabriel Mountains, an Adjustment and Transposition (A-T) factor needs to be applied to debris yields from the study watersheds. The A-T factor can be determined using Table 819.7J by finding the appropriate subfactor for each of the four groups (Parent Material, Soils, Channel Morphology, and Hillside Morphology) and summing the subfactors. This sum is the total A-T factor, and it must be multiplied by the sediment/debris yield.

Once the sediment/debris yield value has been determined based on the unit yield, a bulking factor can be calculated using a series of equations. The first equation provides a translation of the clear-water discharge to a sediment discharge. This clear-water discharge should be developed using a hydrograph method and a hydrologic modeling program, such as HEC-HMS.

$$Q_S = aQ_W^n$$

Q_S = Sediment Discharge (cfs)

Q_W = 100-Year Clear-Water Discharge (cfs)

a = Bulking Constant

For a majority of sand-bed streams, the value of “n” is between 2 and 3. When n=2, the bulking factor is linearly proportional to the clear-water discharge. As for the coefficient “a”, it is determined with the following equation:

$$a = \frac{V_S}{\Delta t \sum Q_W^2}$$

V_S = Total Sediment Volume (cubic feet)

Δt = Computation Time Interval Used In Developing Hydrograph From Hydrologic Model (e.g. HEC-HMS)

Finally, the bulking factor equation is expressed as follows:

$$BF = \frac{Q_W + Q_S}{Q_W} = 1 + aQ_W^{n-1}$$

(g) Recommended Approach For Developing Bulking Factors

A flow chart outlining the recommended bulking factor process is provided in Figure 819.7H, which considers all bulking methods presented in Topic 819.

As shown in Steps 4 and 5 on Figure 819.7H, a bulking factor can be found by:

a. Identifying the type of flow within a watershed and selecting the corresponding bulking factor, or

b. Using one of the agency methods to calculated the bulking factor.

If the type of flow cannot be identified or the project site does not fall within the recommended boundaries from Figure 819.7H, use the LA District Method because it is the most universal given its use of the Adjustment-Transposition factor based on study watershed properties.

Table 819.7J

Adjustment-Transportation Factor Table

	A-T Subfactor				
	0.25	0.20	0.15	0.10	0.05
Parent Material	Subfactor Group 1				
Folding	Severe		Moderate		Minor
Faulting	Severe		Moderate		Minor
Fracturing	Severe		Moderate		Minor
Weathering	Severe		Moderate		Minor
Soils	Subfactor Group 2				
Soils	Non-cohesive		Partly Cohesive		Highly Cohesive
Soil Profile	Minimal Soil Profile		Some Soil Profile		Well-developed Soil Profile
Soil Cover	Much Bare Soil in Evidence		Some Bare Soil in Evidence		Little Bare Soil in Evidence
Clay Colloids	Few Clay Colloids		Some Clay Colloids		Many Clay Colloids
Channel Morphology	Subfactor Group 3				
Bedrock Exposures	Few Segments in Bedrock		Some Segments in Bedrock		Many Segments in Bedrock
Bank Erosion	> 30% of Banks Eroding		10 – 30% of Banks Eroding		< 10% of Banks Eroding
Bed and Bank Materials	Non-cohesive Bed and Banks		Partly Cohesive Bed and Banks		Mildly Cohesive Bed and Banks
Vegetation	Poorly Vegetated		Some Vegetation		Much Vegetation
Headcutting	Many Headcuts		Few Headcuts		No Headcutting
Hillslope Morphology	Subfactor Group 4				
Rills and Gullies	Many and Active		Some Signs		Few Signs
Mass Movement	Many Scars Evident		Few Signs Evident		No Signs Evident
Debris Deposits	Many Eroding Deposits		Some Eroding Deposits		Few Eroding Deposits
The A-T Factor is the sum of the A-T Subfactors from all 4 Subfactor Groups.					

Figure 819.7H

Recommended Bulking Factor Selection Process

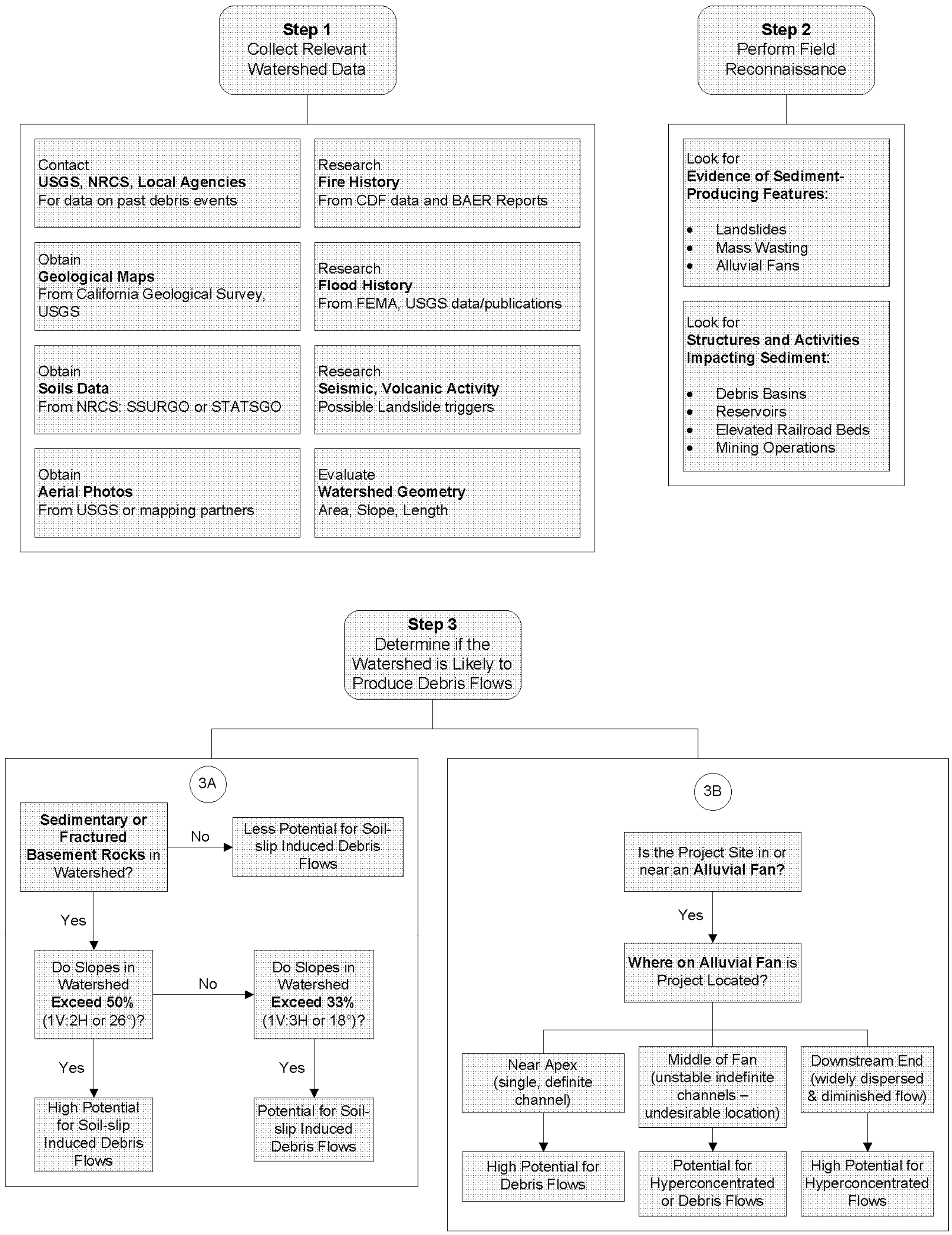
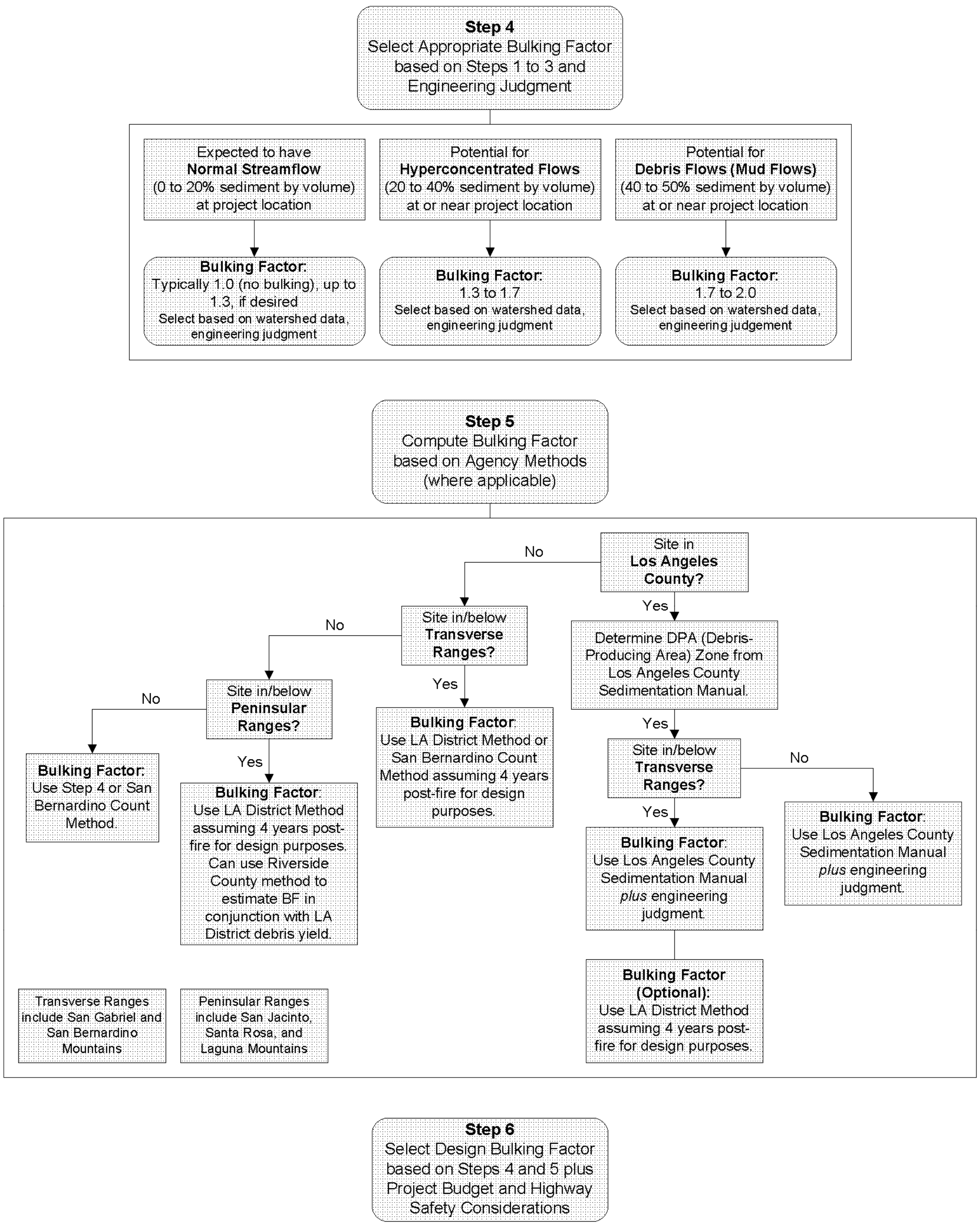


Figure 819.7H

Recommended Bulking Factor Selection Process (Cont'd)



CHAPTER 820
CROSS DRAINAGE

Topic 821 - General

Index 821.1 - Introduction

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some other type of flow obstruction.

In addition to the hydraulic function, a culvert must carry construction and highway traffic and earth loads. Culvert design, therefore, involves both hydraulic and structural design. This section of the manual is basically concerned with the hydraulic design of culverts. Both the hydraulic and structural designs must be consistent with good engineering practice and economics. An itemized listing of good drainage design objectives and economic factors to be considered are listed in Index 801.4. Information on strength requirements, height of fill tables, and other physical characteristics of alternate culvert shapes and materials may be found in Chapter 850, Physical Standards.

More complete information on hydraulic principles and engineering techniques of culvert design may be found in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts". Key aspects of culvert design and a good overview of the subject are more fully discussed in the AASHTO Highway Drainage Guidelines.

Structures measuring more than 20 feet along the roadway centerline are conventionally classified as bridges, assigned a bridge number, and maintained and inspected by the Division of Structures. However, some structures classified as bridges are designed hydraulically and structurally as culverts. Some examples are certain multi-barreled box culverts and arch culverts. Culverts, as distinguished from bridges, are usually covered with embankment and have structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.

Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economic and hydraulic efficiency, culverts should be designed to operate with the inlets submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements of the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations.

Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is relatively small, the cumulative cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the cost of maintaining highway drainage features is substantial, and culvert maintenance is a large share of these costs. Improved service to the public and a reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

821.2 Hydrologic Considerations

Before the hydraulic design of a culvert or bridge can begin, the design discharge, the quantity (Q) of water in cubic feet per second, that the facility may reasonably be expected to convey must be estimated. The most important step is to establish the appropriate design storm or flood frequency for the specific site and prevailing conditions. Refer to Chapter 810, Hydrology and specifically Topics 818 and 819 for useful information on hydrological analysis methods and considerations.

When empirical methods are used to estimate the peak rate of runoff, design Q, for important culverts, it is recommended that at least two methods be tried. By comparing results a more reliable discharge estimate for the drainage basin may be obtained. This is more important for large basins having areas in excess of 320 acres than for small basins.

821.3 Selection of Design Flood

As discussed in Index 818.2, there are two recognized alternatives to selecting the design flood frequency (probability of exceedance) in the hydraulic design of bridges and culverts. They are:

- By policy - using a preselected recurrence interval.
- By analysis - using the recurrence interval that is most cost effective and best satisfies the specific site conditions and associated risks.

Although either of these alternatives may be used exclusive of the other, in actual practice both alternatives are often considered and used jointly to select the flood frequency for hydraulic design. For culverts and bridges, apply the following general rules for first consideration in the process for ultimate selection of the design flood.

(1) *Bridges.* The basic rule for the hydraulic design of bridges (but not including those culvert structures that meet the definition of a bridge) is that they should pass a 2 percent probability flood (50-year). Freeboard, vertical clearance between the lowest structural member and the water surface elevation of the design flood, sufficient to accommodate the effects of bedload and debris should be provided. Alternatively, a waterway area sufficient to pass the 1 percent probability flood without freeboard should be provided. Two feet of freeboard is often assumed for preliminary bridge designs. The effects of bedload and debris should be considered in the design of the bridge waterway.

(2) *Culverts.* There are two primary design frequencies that should be considered:

  - A 10% probability flood (10-year) without causing the headwater elevation to rise above the inlet top of the culvert and,
  - A 1% probability flood (100-year) with-out headwaters rising above an elevation that would cause objectionable backwater depths or outlet velocities.

The designer must use discretion in applying the above criteria. Design floods selected on this basis may not be the most appropriate for specific project site locations or conditions. The cost of providing facilities to pass peak discharges suggested by these criteria need to be balanced against potential damage to the highway and adjacent properties upstream and downstream of the site. The selection of a design flood with a lesser or greater peak discharge may be warranted and justified by economic analysis. A more frequent design flood than a 4% probability of exceedance (25-year) should not be used for the hydraulic design of culverts under freeways and other highways of major importance. Alternatively, where predictive data is limited, or where the risks associated with drainage facility failure are high, the greatest flood of record or other suitably large event should be evaluated by the designer.

When channels or drainage facilities under the jurisdiction of local flood control agencies or Corps of Engineers are involved, the design flood must be determined through negotiations with the agencies involved.

821.4 Headwater and Tailwater

(1) *Headwater.* The term, headwater, refers to the depth of the upstream water surface measured from the invert of the culvert entrance. Any culvert which constricts the natural stream flow will cause a rise in the upstream water surface.

It is not always economical or practical to utilize all the available head. This applies particularly to situations where debris must pass through the culvert, where a headwater pool cannot be tolerated, or where the natural gradient is steep and high outlet velocities are objectionable.

The available head may be limited by the fill height, damage to the highway facility, or the effects of ponding on upstream property. The extent of ponding should be brought to the attention of all interested functions, including Project Development, Maintenance, and Right of Way.

Full use of available head may develop some vortex related problems and also develop objectionable velocities resulting in abrasion of the culvert itself or in downstream erosion. In most cases, provided the culvert is not flowing under pressure, an increase in the culvert size does not appreciably change the outlet velocities.

(2) *Tailwater.* The term, tailwater, refers to the water located just downstream from a structure. Its depth or height is dependent upon the downstream topography and other influences. High tailwater could submerge the culvert outlet.

821.5 Effects of Tide and Wind

Where the tailwater elevation is controlled by tides, special studies will normally be required to determine the tailwater stage consistent with the design storm frequency of the facility. The effects of wind and flood discharges must be considered in conjunction with predicted tide stages. Where necessary, backflow protection should be provided in the form of flap gates. Refer to Indexes 838.3 and 838.5(2) for further discussion of this subject.

Topic 822 - Debris Control

822.1 Introduction

Debris, if allowed to accumulate either within a culvert or at its inlet, can adversely affect the hydraulic performance of the facility. Damage to the roadway and to upstream property may result from debris obstructing the flow into the culvert. Coordination with district maintenance forces can help in identifying areas with high debris potential and in setting requirements for debris removal where necessary.

The use of any device that can trap debris must be thoroughly examined prior to its use. In addition to the more common problem of debris accumulation at the culvert entrance, the use of safety end grates or other appurtenances can also lead to debris accumulation within the culvert at the outlet end. Evaluation of this possibility, and appropriate preventive action, must be made if such end treatment is proposed.

822.2 Debris Control Methods

There are two methods of handling debris:

(1) *Passing Through Culvert.* If economically feasible, culverts should be designed to pass debris. Culverts which pass debris often have a higher construction cost. On the other hand, retaining solids upstream from the entrance by means of a debris control structure often involves substantial maintenance cost and could negatively affect fish passage. An economic comparison which includes evaluation of long term maintenance costs should be made to determine the most reasonable and cost effective method of handling.

(2) *Interception.* If it is not economical to pass debris, it should be retained upstream from the entrance by means of a debris control structure or the use of a debris basin when the facility is located in the vicinity of alluvial fans.

If drift and debris are retained upstream, a riser or chimney may be required. This is a vertical extension to the culvert which provides relief when the main entrance is plugged. The increased head should not be allowed to develop excessive velocities or cause pressure which might induce leakage in the culvert.

If debris control structures are used, access must be provided for maintenance equipment to reach the site. This can best be handled by coordination and field review with district maintenance staff. Details of a pipe riser with debris rack cage are shown on Standard Plan D93C. See FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures" for further information.

The use of an upstream debris basin and downstream concrete lined channels, has often been used by Local Agencies for managing flood flows on alluvial fans in urbanized areas. Experience has shown that this approach is effective, however, the costs of building and maintaining such facilities is high with a potential for sediment inflows greater than anticipated.

The District Hydraulics Engineer should be consulted if a debris basin is being considered for interception in the vicinity of an alluvial fan.

822.3 Economics

Debris problems do not occur at all suspected locations. It is often more economical to construct debris control structures after problems develop. An assessment of potential damage due to debris clogging if protection is not provided should be the basis of design.

822.4 Classification of Debris

In order to properly determine methods for debris control, an evaluation of the characteristics of debris within flood flows must be made. Debris can be either floating, suspended in the flood flow, or dragged/rolled along the channel bottom. Typically, a flood event will deposit debris from all of these types.

The FHWA Hydraulic Engineering Circular No. 9 contains a debris classification system to aid the designer in selecting the appropriate type of debris control structure.

822.5 Types of Debris Control Structures

The FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures", shows types of debris control structures and provides a guide for selecting the type of structure suitable for various debris classifications.

Topic 823 - Culvert Location

823.1 Introduction

The culvert usually should be located so that the thalweg of the stream to be accommodated, approaches and exits at the approximate centerline of the culvert. However, for economic reasons, as a general rule, small skews should be eliminated, moderate skews retained and large skews reduced.

Since the culvert typically acts as a constriction, local velocities will increase through the barrel and in the vicinity of the outlet. The location and design must be also sensitive to the environment (fish passage etc).

As a general rule, flood waters should be conducted under the highway at first opportunity minimizing scour of embankment and entrapment of debris. Therefore, culverts should be placed at each defined swale to limit carryover of drainage from one watershed to another.

823.2 Alignment and Slope

The ideal culvert placement is on straight alignment and constant slope. Variations from a straight alignment should be only to accommodate unusual conditions. Where conditions require deviations from the tangent alignment, abrupt changes in direction or slope should be avoided in order to maintain the hydraulic efficiency, and avoid excessive maintenance. Angle points may be permissible in the absence of abrasives in the flow; otherwise, curves should be used. When angle points are unavoidable, maintenance access may be necessary. See Index 838.5 for manhole location criteria.

Curvature in pipe culverts is obtained by a series of angle points. Whenever conditions require these angle points in culvert barrels, the number of angle points must be specified either in the plans or in the special provisions. The angle can vary depending upon conditions at the site, hydraulic requirements, and purpose of the culvert. The angle point requirement is particularly pertinent if there is a likelihood that structural steel plate pipe will be used. The structural steel plate pipe fabricator must know what the required miters are in order for the plates to be fabricated satisfactorily. Manufacturers' literature should be consulted to be sure that what is being specified can be fabricated without excessive cost.

Ordinarily the grade line should coincide with the existing streambed. Deviations from this practice are permissible under the following conditions:

(a) On flat grades where sedimentation may occur, place the culvert inlet and outlet above the streambed but on the same slope. The distance above the streambed depends on the size length and amount of sediment anticipated.

If possible, a slope should be used that is sufficient to develop self-cleaning velocities.

(b) Under high fills, anticipate greater settlement under the center than the sides of the fill. Where settlement is anticipated, provisions should be made for camber.

(c) In steep sloping areas such as on hillsides, the overfill heights can be reduced by designing the culvert on a slope flatter than natural slope. However, a slope should be used to maintain a velocity sufficient to carry the bedload. A spillway or downdrain can be provided at the outlet. Outlet protection should be provided to prevent undermining. For the downdrain type of installation, consideration must be given to anchorage. This design is appropriate only where substantial savings will be realized.

Topic 824 - Culvert Type Selection

824.1 Introduction

A culvert is a hydraulically short conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

824.2 Shape and Cross Section

(1) Numerous cross-sectional shapes are available. The most commonly used shapes include circular, box (rectangular), elliptical, pipe-arch, and arch. The shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance.

(2) Multiple Barrels. In general, the spacing of pipes in a multiple installation, measured between outside surfaces, should be at least half the nominal diameter with a minimum of 2 feet.

See Standard Plan D89 for multiple pipe headwall details.

Additional clearance between pipes is required to accommodate flared end sections. See Standard Plans, D94A & B for width of flared end sections.

Topic 825 - Hydraulic Design of Culverts

825.1 Introduction

After the design discharge, (Q), has been estimated, the conveyance of this water must be investigated. This aspect is referred to as hydraulic design.

The highway culvert is a special type of hydraulic structure. An exact theoretical analysis of culvert flow is extremely complex because the flow is usually non-uniform with regions of both gradually varying and rapidly varying flow. Hydraulic jumps often form inside or downstream of the culvert barrel. As the flow rate and tailwater elevations change, the flow type within the barrel changes. An exact hydraulic analysis therefore involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic studies.

An extensive hydraulic analysis is usually impractical and not warranted for the design of most highway culverts. The culvert design procedures presented herein and in the referenced publications are accurate, in terms of head, to within plus or minus 10 percent.

825.2 Culvert Flow

The types of flow and control used in the design of highway culverts are:

- Inlet Control - Most culverts operate under inlet control which occurs when the culvert barrel is capable of carrying more flow than the inlet will accept. Supercritical flow is usually encountered within the culvert barrel. When the outlet is submerged under inlet control, a hydraulic jump will occur within the barrel.

- Outlet Control - Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet will accept. Culverts under outlet control generally function with submerged outlets and subcritical flow within the culvert barrel. However, it is possible for the culvert to function with an unsubmerged outlet under outlet control where flow passes through critical depth in the vicinity of the outlet.

For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross sectional area of the culvert, inlet geometry, and elevation of headwater at entrance are of primary importance. Outlet control involves the additional consideration of the tailwater elevation of the outlet channel and the slope, roughness and length of the culvert barrel. A discussion of these two types of control with charts for selecting a culvert size for a given set of conditions is included in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts."

825.3 Computer Programs

Numerous calculator and computer programs are available to aid in the design and analysis of highway culverts. The major advantages of these programs over the traditional hand calculation method are:

- Increased accuracy over charts and nomographs.
- Rapid comparison of alternative sizes and inlet configurations.

Familiarity with culvert hydraulics and traditional methods of solution is necessary to provide a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities of hydraulic design computer programs.

The hydraulic design calculator and computer programs available from the FHWA are more fully described in HDS No. 5, "Hydraulic Design of Highway Culverts."

The HY8 culvert hydraulics program provides interactive culvert analysis. Given all of the appropriate data, the program will compute the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined culverts.

The logic of HY8 involves calculating the inlet and outlet control headwater elevations for the given flow. The elevations are then compared and the larger of the two is used as the controlling elevation. In cases where the headwater elevation is greater then the top elevation of the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the surcharge over the roadway. In the cases where the culvert is not full for any part of its length, open channel computations are performed.

825.4 Coefficient of Roughness

Suggested Manning's n values for culvert design are given in Table 852.1.

Topic 826 - Entrance Design

826.1 Introduction

The size and shape of the entrance are among the factors that control the level of ponding at the entrance. Devices such as rounded or beveled lips and expanded entrances help maintain the velocity of approach, increase the culvert capacity, and may lower costs by permitting a smaller sized culvert to be used.

The inherent characteristics of common entrance treatments are discussed in Index 826.4. End treatment on large culverts is an important consideration. Selecting an appropriate end treatment for a specific type of culvert and location requires the application of sound engineering judgment.

The FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts" combines culvert design information previously contained in HEC No. 5, No. 10, and No. 13. The hydraulic performance of various entrance types is described in HDS No. 5.

826.2 End Treatment Policy

The recommended end treatment for small culverts is the prefabricated flared end section. For safety, aesthetic, and economic reasons, flared end sections should be used at both entrance and outlet whenever feasible instead of headwalls.

End treatment, either flared end section or headwall, is required for circular culverts 60 inches or more in diameter and for pipe arches of equivalent size.

826.3 Conventional Entrance Designs

The inlet edge configuration is one of the prime factors influencing the hydraulic performance of a culvert operating in inlet control. The following entrance types are frequently used.

(1) *Projecting Barrel.* A thin edge projecting inlet can cause a severe contraction of the flow. The effective cross sectional area of the barrel may be reduced to about one half the actual available barrel area.

The projecting barrel has no end treatment and is the least desirable hydraulically. It is economical but its appearance is not pleasing and use should be limited to culverts with low velocity flows where head conservation, traffic safety, and appearance are not important considerations.

Typical installations include an equalizer culvert where ponding beyond the control of the highway facility occurs on both sides of the highway or where the flow is too small to fill the minimum culvert opening.

The projecting entrance inhibits culvert efficiency. In some situations, the outlet end may project beyond the fill, thus providing security against erosion at less expense than bank protection work.

Projecting ends may prove a maintenance nuisance, particularly when clearance to right of way fence is limited.

(2) *Flared End Sections.* This end treatment provides approximately the same hydraulic performance as a square-edge headwall and is used to retain the embankment, improve the aesthetics, and enhance safety. Because prefabricated flared end sections provide better traffic safety features and are considered more attractive than headwalls they are to be used instead of headwalls whenever feasible.

Details of prefabricated flared end sections for circular pipe in sizes 12 inches through 84 inches in diameter and pipe arches of equivalent size are shown on Standard Plans D94A & B.

(3) *Headwalls and Wingwalls.* This end treatment may be required at the culvert entrance for the following reasons:

- To improve hydraulic efficiency.
- To retain the embankment and reduce erosion of slopes.
- To provide structural stability to the culvert ends and serve as a counterweight to offset buoyant or uplift forces.

(4) *Rounded Lip.* This treatment costs little, smoothes flow contraction, increases culvert capacity, and reduces the level of ponding at the entrance. The box culvert and pipe headwall standard plans include a rounded lip. The rounded lip is omitted for culverts less than 48 inches in diameter; however, the beveled groove end of concrete pipe at the entrance produces an effect similar to that of a rounded lip.

(5) *Mitered End.* A mitered culvert end is formed when the culvert barrel is cut to conform with the plane of the embankment slope. Mitered entrances are not to be used. They are hydraulically less efficient than either flared end sections or headwalls, and they are structurally unstable.

(6) *Entrance Risers.* At a location where the culvert would be subject to plugging, a vertical pipe riser should be considered. Refer to Index 822.2 for discussion on debris-control structures.826.4 Improved Inlet Designs

826.4 Improved Inlet Designs

Entrance geometry refinements can be used to reduce the flow contraction at the inlet and increase the capacity of culverts operating under inlet control without increasing the headwater depth. The following entrance types improve culvert inlet performance and can be provided at reasonable cost.

(1) *Expanded Entrances.* Headwalls with straight flared wingwalls or warped wingwalls offer a more highly developed entrance appropriate for large culverts, regardless of type or shape of barrel. The effect of such entrances can be approximated more economically by a shaped entrance using air blown mortar, concreted riprap, sacked concrete or slope paving.

Straight flared wingwalls and warped wingwalls aid in maintaining the approach velocity, align and guide drift, and funnel the flow into the culvert entrance. To insure enough velocity to carry drift and debris through the culvert or increase the velocity and thereby increase the entrance capacity, a sloping drop down apron at the entrance may be used. To minimize snagging drift, the standard plans require wingwalls to be flush with the culvert barrel. The flare angle may range from 30 to 75 degrees; the exact angle is based on the alignment of the approach channel banks and not the axis of the culvert. Greater efficiency is obtained when the top of the wingwall is the same elevation as the headwall.

Whether warped or straight flared wingwalls are used depends on the shape of the approach channel. Straight flared wingwalls are appropriate for well defined channels with steep banks. Warped wingwalls are more suited to shallow trapezoidal approach channels.

Usually it is more economical to transition between the stream section and the culvert by means of straight flared wingwalls or warped wingwalls than to expand the culvert barrel at entrance. For a very wide channel, this transition may be combined with riprap, dikes, or channel lining extending upstream to complete the transition.

(2) *Transitions.* Elaborate transitions and throated openings for culverts may be warranted in special cases. Generally a highly developed entrance is unnecessary if the shape of the culvert fits the approach channel. In wide flat channels where ponding at entrance must be restricted, a wide shallow structure or multiple conduit should be used if drift and debris are not a problem.

Throated or tapered barrels at entrance are more vulnerable to clogging by debris. They are not economical unless they are used for corrective measures; for example, where there is a severe restriction in right of way width and it is necessary to increase the capacity of an existing culvert structure.

For further information refer to HEC-9, "Debris-Control Structures" and HDS 5, "Hydraulic Design of Highway Culverts"

Topic 827 - Outlet Design

827.1 General

The outlet velocity of highway culverts is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet.

The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. The shape and size of a culvert seldom have a significant effect on the outlet velocity. When the outlet velocity is believed to be excessive and it cannot be satisfactorily reduced by adjusting the slope or barrel roughness, it may be necessary to use some type of outlet protection or energy dissipator. A method of predicting and analyzing scour conditions is given in the FHWA publication "Scour at Culvert Outlets in Mixed Bed Materials", FHWA/RD - 82/011.

When dealing with erosive velocities at the outlet, the effect on downstream property should be evaluated.

827.2 Embankment Protection

Improved culvert outlets are designed to restore natural flow conditions downstream. Where erosion is to be expected, corrective measures such as bank protection, vertical flared wingwalls, warped wingwalls, transitions, and energy dissipators may be considered. See Chapter 870, "Channel and Shore Protection-Erosion Control", FHWA Hydraulic Engineering Circulars No. 11, "Design of Riprap Revetment", No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", and No. 15, "Design of Roadway Channels with Flexible Linings", and "Hydraulic Design of Stilling Basins and Energy Dissipators", Engineering Monograph No. 25 by the U. S. Department of Interior, Bureau of Reclamation, 1964 (revised 1978). HY-8, within the Hydrain Integrated Computer Program System, provides designs for energy dissipators and follows the HEC-14 method for design.

Culvert outlet design should provide a transition for the 100-year flood or design event from the culvert outlet to a section in the natural channel where natural stage, width, and velocity will be restored, or nearly so, with consideration of stability and security of the natural channel bed and banks against scour.

If an outfall structure is required for transition, typically it will not have the same design as the entrance.

Wingwalls, if intended for an outlet transition (expansion), generally should not flare at an angle (in degrees) greater than 150 divided by the outlet velocity in feet per second. However, transition designs fall into two general categories: those applicable to culverts in outlet control (subcritical flow) or those applicable to culverts in inlet control (supercritical). The procedure outlined in HEC-14 for subcritical flow expansion design should also be used for supercritical flow expansion design if the culvert exit Froude number (Fr) is less than 3, if the location where the flow conditions desired is within 3 culvert diameters of the outlet, and if the slope is less than 10 percent. For supercritical flow expansions outside these limits, the energy equation can be used to determine flow conditions leaving the transition.

Warped endwalls can be designed to fit trapezoidal or U-shaped channels, as transitions for moderate-to-high velocity (10 feet per second – 18 feet per second).

For extreme velocity (exceeding 18 feet per second) the transition can be shortened by using an energy-dissipating structure.

Topic 828 - Diameter and Length

828.1 Introduction

From a maintenance point of view the minimum diameter of pipe and the distance between convenient cleanout access points are important considerations.

The following instructions apply to minimum pipe diameter and the length of pipe culvert.

828.2 Minimum Diameter

The minimum diameter for cross culverts under the roadway is 18 inches. For other than cross pipes, the minimum diameter is 12 inches. For maintenance purposes, where the slope of longitudinal side drains is not sufficient to produce self-cleaning velocities, pipe sizes of 18 inches or more in diameter should be considered.

The minimum diameter of pipe to be used is further determined by the length of pipe between convenient cleanout access points. If pipe runs exceed 100 feet between inlet and outlet, or intermediate cleanout access, the minimum diameter of pipe to be used is 24 inches. When practicable, intermediate cleanout points should be provided for runs of pipe 24 inches in diameter that exceed 300 feet in length.

If a choice is to be made between using 18-inch diameter pipe with an intermediate cleanout in the highway median or using 24-inch diameter pipe without the median access, the larger diameter pipe without the median access is preferred.

828.3 Length

The length of pipe culvert to be installed is determined as follows:

(a) Establish a theoretical length based on slope stake requirements making allowance for end treatment.

(b) Adjust the theoretical length for height of fill by applying these rules:

- For fills 12 feet or less, no adjustment is required.
- For fills higher than 12 feet, add 1 foot of length at each end for each 10 foot increment of fill height or portion thereof. The additional length should not exceed 6 feet on each end.
- In cases of high fills with benches, the additional length is based on the height of the lowest bench.

(c) Use the nearest combination of commercial lengths which equal or exceed the length obtained in (b) above.

Topic 829 - Special Considerations

829.1 Introduction

In addition to the hydraulic design, other factors must be considered to assure the integrity of culvert installations and the highway.

829.2 Bedding and Backfill

The height of overfill a culvert will safely sustain depends upon foundation conditions, method of installation, and its structural strength and rigidity.

Uniform settlement under both the culvert and the adjoining fill will not overstress flexible and segmental rigid culverts. Unequal settlement, however, can result in distortion and shearing action in the culvert. For rigid pipes this could result in distress and disjointing of the pipe. A flexible culvert accommodates itself to moderate unequal settlements but is also subject to shearing action. Monolithic culverts can tolerate only a minimal amount of unequal settlement, and require favorable foundation conditions. Any unequal settlement would subject a monolithic culvert to severe shear stresses.

(1) Foundation Conditions. A slightly yielding foundation under both the culvert and adjoining fill is the foundation condition generally encountered. The maximum height of cover tables given in Chapter 850 are based on this foundation condition.

Unyielding foundation conditions can produce high stresses in the culverts. Such stresses may be counteracted by subexcavation and backfill.

The Standard Plans show details for shaped, sand, and soil cement bedding treatments.

Foundation materials capable of supporting pressures between 1.0 tons per square foot and 8.0 tons per square foot are required for culverts with cast-in-place footing or inverts, such as reinforced concrete boxes, arches, and structural plate arches. When culvert footing pressures exceed 1.5 tons per square foot or the diameter or span exceeds 10 feet, a geology report providing a log of test boring is required.

Adverse foundation and backfill conditions may require a specially designed structure. The allowable overfill heights for concrete arches, structural plate arches, and structural plate vehicular undercrossings are based on existing soil withstanding the soil pressures indicated on the Standard Plans. A foundation investigation should be made to insure that the supporting soils withstand the design soil pressures for those types of structures.

(2) Method of Installation. Under ordinary conditions, the methods of installation described in the Standard Specifications and shown on the Standard Plans should be used. For any predictable settlement, provisions for camber should be made.

Excavation and backfill details for circular concrete pipe, reinforced box and arch culverts, and corrugated metal pipe and arch culverts are shown on Standard Plans A62-D, A62DA, A62-E, and A62-F respectively.

(3) Height of Cover. There are several alternative materials from which acceptable culverts may be made. Tables of maximum height of cover recommended for the more frequently used culvert shapes, sizes, corrugation

configurations, and types of materials are given in Chapter 850. Not included, but covered in the Standard Plans, are maximum earth cover for reinforced concrete box culverts, reinforced concrete arches, and structural plate vehicular undercrossing.

For culverts where overfill requirements exceed the limits shown on the tables a special design must be prepared. Special designs are to be submitted to the Division of Structures for review, or the Division of Structures may be directly requested to prepare the design.

Under any of the following conditions, the Division of Structures is to prepare the special design:

- Where foundation material will not support footing pressure shown on the Standard Plans for concrete arch and structural plate vehicular undercrossings.
- Where foundation material will not support footing pressures shown in the Highway Design Manual for structural plate pipe arches or corrugated metal pipe arches.
- Where a culvert will be subjected to unequal lateral pressures, such as at the toe of a fill or adjacent to a retaining wall.

Special designs usually require that a detailed foundation investigation be made.

(4) *Minimum Cover.* When feasible, culverts should be buried at least 1 foot. For construction purposes, a minimum cover of 6 inches greater than the thickness of the structural cross section is desirable for all types of pipe. The minimum thickness of cover for various type culverts under rigid or flexible pavements is given in Table 856.5.

829.3 Piping

Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a hollow similar to a pipe. Fine soil particles are washed out freely along the hollow and the erosion inside the fill may ultimately cause failure of the culvert or the embankment.

The possibility of piping can be reduced by decreasing the velocity of the seepage flow. This can be reduced by providing for watertight joints. Therefore, if piping through joints could become a problem, consideration should be given to providing for watertight joints.

Piping may be anticipated along the entire length of the culvert when ponding above the culvert is expected for an extended length of time, such as when the highway fill is used as a detention dam or to form a reservoir. Headwalls, impervious materials at the upstream end of the culvert, and anti-seep or cutoff collars increase the length of the flow path, decrease the hydraulic gradient and the velocity of flow and thus decreases the probability of piping developing. Anti-seep collars usually consist of bulkhead type plate or blocks around the entire perimeter of the culvert. They may be of metal or concrete, and, if practical, should be keyed into impervious material.

Piping could occur where a culvert must be placed in a live stream, and the flow cannot be diverted. Under these conditions watertight joints should be specified.

829.4 Joints

The possibility of piping being caused by open joints in the culvert barrel may be reduced through special attention to the type of pipe joint specified. For a more complete discussion of pipe joint requirements see Index 854.1.

The two pipe joint types specified for culvert installations are identified as "standard" and "positive". The "standard" joint is adequate for ordinary installations and "positive" joints should be specified where there is a need to withstand soil movements or resist disjointing forces. Corrugated metal pipe coupling band details are shown on Standard Plan sheets D97A through D97G and concrete pipe joint details on sheet D97H.

If it is necessary for "standard" or "positive" joints to be watertight they must be specifically specified as such. Rubber "O" rings or other resilient joint material provides the watertight seal. Corrugated metal pipe joints identified as "downdrain" are watertight joint systems with a tensile strength specification for the coupler.

829.5 Anchorage

Refer to Index 834.4(5) for discussion on anchorage for overside drains.

Reinforced concrete pipe should be anchored and have positive joints specified if either of the following conditions is present:

(a) Where the pipe diameter is 60 inches or less, the pipe slope is 33 percent or greater, and the fill over the top of the pipe less than 1.5 times the outside diameter of the pipe measured perpendicular to the slope.

(b) Where the pipe diameter is greater than 60 inches and the pipe slope is 33 percent or greater, regardless of the fill over the top of the pipe.

Where the slopes have been determined by the geotechnical engineer to be potentially unstable, regardless of the slope of the pipe, as a minimum, the pipes shall have positive joints. Alternative pipes/anchorage systems shall be investigated when there is a potential for substantial movement of the soil.

Where anchorage is required, there should be a minimum of 18 inches cover measured perpendicular to the slope.

Typically buried flexible pipe with corrugations on the exterior surface will not require anchorage, however, a special detail will be required for plastic pipe without corrugations on the exterior surface.

829.6 Irregular Treatment

(1) Junctions. (Text Later)

(2) Bends. (Text Later)

829.7 Siphons and Sag Culverts

(1) General Notes. There are two kinds of conduits called siphons: the true siphon and the inverted siphon or sag culvert. The true siphon is a closed conduit, a portion of which lies above the hydraulic grade line. This results in less than atmospheric pressure in that portion. The sag culvert lies entirely below the hydraulic grade line; it operates under pressure without siphonic action.

Under the proper conditions, there are hydraulic and economic advantages to be obtained by using the siphon principle in culvert design.

(2) Sag Culverts. This type is most often used to carry an irrigation canal under a highway when the available headroom is insufficient for a normal culvert. The top of a sag culvert should be at least 4.5 feet below the finished grade where possible, to ensure against damage from heavy construction equipment. The culvert should be on a straight grade and sumps provided at each end to facilitate maintenance. Sag culverts should not be used:

(a) When the flow carries trash and debris in sufficient quantity to cause heavy deposits,

(b) For intermittent flows where the effects of standing water are objectionable, or

(c) When any other alternative is possible at reasonable cost.

(3) Types of Conduit. Following are two kinds of pipes used for siphons and sag culverts to prevent leakage:

(a) Reinforced Concrete Pipe - Reinforced concrete pipe with joint seals is generally satisfactory. For heads over 6 m, special consideration should be given to hydrostatic pressure.

(b) Corrugated Metal Pipe - corrugated metal pipe must be of the thickness and have the protective coatings required to provide the design service life. Field joints must be watertight. The following additional treatment is recommended.

- When the head is more than 10 feet and the flow is continuous or is intermittent and of long duration, pipe fabricated by riveting, spot welding or continuous helical lockseam should be soldered.

  Pipe fabricated by a continuous helical welded seam need not be soldered.

- If the head is 10 feet or less and the flow is intermittent and lasts only a few days, as in storm flows, unsoldered seams are permissible.

829.8 – Currently Not In Use

829.9 Dams

Typically, proposed construction which is capable of impounding water to the extent that it meets the legal definition of a dam must be approved by the Department of Water Resource (DWR), Division of Safety of Dams. The legal definition is described in Sections 6002 and 6003 of the State Water Code. Generally, any facility 25 feet or more in height or capable of impounding 50 acre-feet or more would be considered a dam. However, any facility 6 feet or less in height, regardless of capacity, or with a storage capacity of not more than 15 acre-feet, regardless of height, shall not be considered a dam. Additionally, Section 6004 of the State Water Code states "... and no road or highway fill or structure ... shall be considered a dam." Therefore, except for large retention or detention facilities there will rarely be the need for involvement by the DWR in approval of Caltrans designs.

Although most highway designs will be exempt from DWR approval, caution should always be exercised in the design of high fills that could impound large volumes of water. Even partial plugging of the cross drain could lead to high pressures on the upstream side of the fill, creating seepage through the fill and/or increased potential for piping.

The requirements for submitting information to the FHWA Division Office in Sacramento as described in Index 805.6 are not affected by the regulations mentioned above.

829.10 Reinforced Concrete Box Modifications

(1) *Extensions.* Where an existing box culvert is to be lengthened, it is essential to perform an on-site investigation to verify the structural integrity of the box. If signs of distress are present, the Division of Structures must be contacted prior to proceeding with the design.

(2) *Additional Loading.* When significant additional loading is proposed to be added to an existing reinforced concrete box culvert the Division of Structures must be contacted prior to proceeding with the design. Overlays of less than 6 inches in depth, or widenings that do not increase the per unit loading on the box are not considered to be significant. Designers should also check the extent that previous projects might have increased loading on box culverts, even if the current project is not adding a significant amount of loading.

(2) Gutter Flow

- Limit water spread to Table 831.3
- Maximize interception of gutter flow above superelevation transitions (see Index 837.3)

(3) Sag Areas

- Limit pond duration and depth (see Topic 833)

(4) Overtopping

- Avoid overtopping at cross culverts using appropriate freeboard and/or headwater elevation (see Topic 821)

Where suitable measures cannot be implemented to address conditions such as those identified above, or an identified existing problem area, coordination should be made with the Safety Review Committee per Index 110.7.

831.5 Computer Programs

There are many computer programs available to aid highway design engineers with estimating runoff and ensuing hydraulic design and analysis of roadway drainage facilities.

Refer to Table 808.1 for guidance on selecting appropriate software programs for specific analysis needs.

Familiarity with the fundamentals of hydraulics and traditional methods of solution are necessary to assure that the results obtained are reasonable. There is a tendency for inexperienced engineers to accept computer output as valid without verifying the reasonableness of input and output data.

Topic 832 - Hydrology

832.1 Introduction

The philosophy and principles of hydrology are discussed in Chapter 810. Additional information on methods of estimating storm runoff may be found in FHWA's HEC 22.

832.2 Rational Method

With few exceptions, runoff estimates for roadway drainage design are made by using Rational Methods described under Index 819.2(1). In order to make use of these methods, information on the intensity, duration, and frequency of rainfall for the locality of the project must be established. Refer to Index 815.3(3) for further information on precipitation intensity-duration-frequency (IDF) curves that have been developed for many locations in California.

832.3 Time of Concentration

Refer to Index 816.6 for information on time of concentration.

Topic 833 - Roadway Cross Sections

833.1 Introduction

The geometric cross section of the roadway affects drainage features and hydraulic considerations. Cross slope and width of pavement and shoulders as well as other roadway geometry affect the rate of runoff, width of tolerable spread, and hydraulic design considerations. The cross section of drainage features such as, depressed medians, curbs and gutters, dikes, and side ditches is often controlled by an existing roadway geometric cross section or the one selected for new highway construction.

833.2 Grade, Cross Slope and Superelevation

The longitudinal slope or grade is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 301.2 and 302.2 for cross slope. Where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1 percent. The maximum pavement cross slope should be limited to 4 percent. However, exceptions to the design criteria for cross slope in Index 302.2 must be formally approved in accordance with the requirements Index 82.2, "Approvals for Nonstandard Design." For projects where lanes will be added on the inside of divided highways, or when widening an existing “crowned” 2-lane highway to a 4-lane divided highway, consideration should be given to the use of a “tent section” in order to minimize the number of lanes sloping in the same direction. Refer to Index 301.2. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades. Superelevation is discussed in Topic 202. Refer to Index 831.4 for Hydroplaning considerations.

Topic 834 - Roadside Drainage

834.1 General

Median drainage, ditches and gutters, and overside drains are some of the major roadside drainage facilities.

834.2 Median Drainage

(1) *Drainage Across the Median.* When it is necessary for sheet flow to cross flush medians, it should be intercepted by the use of slotted drains or other suitable alternative facilities. See Standard Plan D98-B for slotted drain details.

Where floodwaters are allowed to cross medians, designers must consider the impacts of railings, barrier or other obstructions to both the depth and spread of flow. Designers should consult their district hydraulic unit for assistance.

(2) *Grade and Cross Slope.* The longitudinal slope or grade for median drainage is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 305.2 and 405.5(4) for standards governing allowable cross slope of medians.

Existing conditions control median grades and attainable cross slope on rehabilitation projects. The flattest desirable grade for earth medians is 0.25 percent and 0.12 percent for paved gutters in the median.

(3) *Erosion.* When velocities are excessive for soil conditions, provisions for erosion control should be provided. See Table 862.2 for recommended permissible velocities for unlined channels.

Economics and aesthetics are to be taken into consideration in the selection of median erosion control measures. Under the less severe conditions, ground covers of natural or synthetic materials which render the soil surface stable against accelerated erosion are adequate. Under the more severe conditions, asphalt or concrete ditch paving may be required.

Whenever median ditch paving is necessary, consideration should be given to the use of cement or lime treatment of the soil. The width treated will depend on the capacity needed to handle the drainage. A depth of 6 inches is generally satisfactory. The amount of cement or lime to be used should be based on laboratory tests of the in-place material to be tested, and normally varies from 6 percent to 10 percent. If a clear or translucent curing compound is used, the completed area is unobtrusive and aesthetically pleasing.

Asphalt concrete ditch paving and soil cement treatments cured with an application of liquid asphalt are highly visible and tend to become unsightly from streaks of eroded material. Cobbles, though effective for erosion control, are not satisfactory in a recovery area for out of control vehicles. See Topic 872 for further discussion on erosion protection and additional types of ditch linings. Erosion control references are given under Index 871.3.

(4) *Economy in Design.* Economy in median drainage can be achieved by locating inlets to utilize available nearby culverts or the collector system of a roadway drainage installation. The inlet capacity can be increased by placing it in a local depression. Use of slotted pipe at sag points where a local depression might be necessary may be an alternative solution to a grate catch basin.

(4) *Inlets in Series.* Where conditions dictate the need for a series of inlets, the recommended minimum spacing should be approximately 20 feet to allow the bypass flow to return to the curb face.

837.4 Hydraulic Design

(1) *Factors Governing Inlet Capacity.* Inlet capacity is a variable which depends on:

(a) The size and geometry of the intake opening,

(b) The velocity and depth of flow and the gutter cross slope just upstream from the intake, and

(c) The amount of depression of the intake opening below the flow line of the waterway.

(2) *General Notes.*

(a) Effect of Grade Profile. The grade profile affects both the inlet location and its capacity. The gutter grade line exerts such an influence that it often dictates the choice of inlet types as well as the gutter treatment opposite the opening. See Index 831.2.

Sag vertical curves produce a flattening grade line which increases the width of flow at the bottom. To reduce ponding and possible sedimentation problems, the following measures should be considered:

- Reduce the length of vertical curve.
- Use a multiple installation consisting of one inlet at the low point and one or more inlets upstream on each side. Refer to HEC 22 for further discussion and design procedures for locating multiple inlets.

  Short sections of slotted or grated line drains on either side of the low point may be used to supplement drop inlets.

(b) Cross Slope for Curbed Gutters. Make the cross slope as steep as possible within limits stated under Index 836.2(2). This concentrates the flow against the curb and greatly increases inlet capacity.

(c) Local Depressions. Use the maximum depression consistent with site conditions; for further details see Index 837.5.

(d) Trash. The curb-opening type inlet, when the first in a series of grate inlets, may intercept trash and improve grate efficiency. In a grade sag, one trash interceptor should be used on each side of the sump.

(e) Design Water Surface Within the Inlet. The crown of the outlet pipe should be low enough to allow for pipe entrance losses plus a freeboard of 0.75 feet between the design water surface and the opening at the gutter intake. This allows sufficient margin for turbulence losses, and the effects of floating trash.

(f) Inlet Floor. The inlet floor should generally have a substantial slope toward the outlet. In a shallow drain system where conservation of head is essential, or any system where the preservation of a nonsilting velocity is necessary, the half round floor shown on the Standard Plan D74C should be used when a pipe continues through the inlet.

(g) Partial Interception. Economies may be achieved by designing inlets for partial interception with the last one or two inlets in series intercepting the remaining flow. See Hydraulic Engineering Circular No. 22.

(3) *Curb-Opening Inlets.* Gutter depressions should be used with curb-opening inlets. The standard gutter depressions for curb-opening inlets, shown on Standard Plan D78 are 0.1 foot and 0.25 foot deep.

Curb-opening inlets are most economical and effective if designed and spaced to intercept only 85 to 90 percent of the flow. This provides for an increased flow depth at the curb face.

Figure 4-11, "Comparison of Inlet Interception Capacity, Slope Variable", and Figure 4-12, "Comparison of Inlet Interception Capacity, Flow Rate Variable" of Hydraulic Engineering Circular No. 22 can be used to obtain interception capacities for various longitudinal grades, cross slopes, and gutter depressions. Charts for determining interception capacities under sump conditions are also available in HEC No. 22.

(4) *Grate Inlets.* The grate inlet interception capacity is equal to the sum of the frontal flow (flow over the grate) interception and the side flow interception. The frontal flow interception will constitute the major portion of the grate interception. In general, grate inlets will intercept all of the frontal flow until a velocity is reached at which water begins to splash over the grate. Charts provided in HEC 22 can be used to compute grate interception capacities for the various grates contained therein. Grate depressions will greatly increase inlet capacity.

The HEC 22 charts neglect the effects of debris and clogging on inlet capacity. In some localities inlet clogging from debris is extensive, while in other locations clogging is negligible. Local experience should dictate the magnitude of the clogging factor, if any, to be applied. In the absence of local experience, design clogging factors of 33 percent for freeways and 50 percent for city streets may be assumed.

Grate type inlets are most economical and effective if designed and spaced to intercept only 75 to 80 percent of the gutter flow.

(5) *Combination Inlets.*

(a) Type GO and GDO Inlet. For design purposes, only the capacity of the grates need be considered. The auxiliary curb opening, under normal conditions, offers little or no increase in capacity; but does act as a relief opening should the grate become clogged. Since the grates of Type GDO are side by side, the inlet capacity is the combined capacity of the two grates.

(b) Type GOL Inlet. The interception capacity of this inlet, a curb-opening upstream of a grate, is equal to the sum of the capacities for the two inlets except that the frontal flow and thus inter-ception capacity of the grate is reduced by interception at the curb opening.

(6) *Pipe Drop Inlets.*

(a) Wall Opening Intake. The standard intake opening 2 feet wide and 8 inches to 12 inches deep provides a capacity of approximately 6.0 CFS when the water surface is 1 foot higher than the lip of the opening. Where the flow is from more than one direction, two or more standard openings may be provided. Higher capacity openings larger than standard may be provided but are of a special design.

(b) Grate Intake. The choice between inlets with a round grate (Types GCP and GMP) and those with a rectangular grate (Type G1) hinges largely on hydraulic efficiency. In a waterway where the greatest depth of flow is at the center, both grates are equally effective. In a waterway where the cross slope concentrates the flow on one side of the grate, the rectangular shape is preferred. For rectangular grates, the charts contained in HEC 22 can be used to compute flow intercept. Round grates (Type 36R) with 0.5 foot of depression develop a capacity of 12 CFS to 15 CFS.

837.5 Local Depressions

(1) *Purpose.* A local depression is a paved hollow in the waterway shaped to concentrate and direct the flow into the intake opening and increases the capacity of the inlet. In a gutter bordered by a curb, it is called a gutter depression.

(2) *Requirements.* Local depressions generally consist of a paved apron or transition of a shape which serves the purpose. Local depressions should meet the following requirements:

(a) Valley Medians. In medians on a grade, the depression should extend a minimum of 10 feet upstream, 6 feet downstream and 6 feet laterally, measured from the edge of the opening. In a grade sag, the depression should extend a minimum of 10 feet on all sides. No median local depression, however should be allowed to encroach on the shoulder area.

The normal depth of depression is 4 inches.

CHAPTER 840 SUBSURFACE DRAINAGE

Topic 841 - General

Index 841.1 - Introduction

Saturation of the structural section or underlying foundation materials is a major cause of premature pavement failures. In addition, saturation can lead to undesirable infiltration into storm drain systems and, where certain soil types are below groundwater, liquefaction can occur due to seismic forces. Subsurface drainage systems designed to rapidly remove and prevent water from reaching or affecting the roadbed are discussed in this chapter.

The solution for subsurface drainage problems often calls for a knowledge of geology and the application of soil mechanics. The Project Engineer should request assistance from Geotechnical Services in the Division of Engineering Services for projects involving cuts, sections depressed below the original ground surface, or whenever the presence of groundwater is likely. Geotechnical Services can also provide assistance related to the design of features to relieve hydrostatic pressure at bridge abutments. The designer should consider the potential for large fluctuations in groundwater levels. Wet periods after several years of drought, or changes to recharge practices can lead to considerable rises in groundwater levels.

For tunnel, structure abutments, or other structure projects which might require relief of hydrostatic pressures, contact Geotechnical Services.

The basis for design will generally be the Geotechnical Design Report. This report will include findings on subsurface conditions and recommendations for design. Refer to Topic 113 for more information on Geotechnical Design Reports.

There are many variables and uncertainties as to the actual subsurface conditions. In general, the more obvious subsurface drainage problems can be anticipated in design; the less obvious are frequently uncovered during construction. Extensive exploration and literature review may be required to obtain the design variables with reasonable accuracy.

841.2 Subsurface (Groundwater) Discharge

Groundwater, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. Subsurface discharge, the rate at which groundwater and infiltration water can be removed depends on the effective hydraulic head and on the permeability, depth, slope, thickness and extent of the water-bearing formation (the aquifer). The discharge can be obtained by analytical methods. Such methods, however, are usually cumbersome and unsatisfactory; field explorations will yield better results.

841.3 Preliminary Investigations

Field investigations may include:

- Soils, geological, and geophysical studies.
- Borings, pits, or trenches to find the elevation, depth, and extent of the aquifer.
- Inspection of cut slopes in the immediate vicinity.
- Measurement of groundwater discharge.

Preliminary investigations should be as thorough as possible, recognizing that further information is sometimes uncovered during construction. Where an existing road is part of new construction, the presence and origin of groundwater is often known or easily detected. Personnel responsible for maintenance of the existing road are an excellent source of such information and should be consulted. Explorations, therefore, are likely to be lesser in scope and cost than explorations for a project on new alignment. In slope stability questions, and other problems of equal importance, an extensive knowledge of subsurface conditions is required. The District should ask for the assistance of Geotechnical Services in such cases.

841.4 Exploration Notes

In general, explorations should be made during the rainy season or after the melting of snow in regions where snow cover is common. An exception would be where seepage occurs from irrigation sources.

Groundwater difficulties frequently stem from water perched on an impermeable layer some distance above the actual water table. Perched water problems can often be solved with horizontal drains. See Index 841.5.

Pumped water supply wells often give unreliable indications of the water table and such data should be used with caution.

841.5 Category of System

Depending upon the scope and complexity of the problem, an appropriate solution may require the installation of one or a combination of different types of subsurface drainage systems. The type of subsurface drainage system initially considered is usually an underdrain.

The standard underdrain is the pipe underdrain. A pipe underdrain consists of a perforated pipe near the bottom of a narrow trench lined with filter fabric and backfilled with permeable material.

Pipe underdrains are discussed in more detail under Topic 842.

"French Drains" have proven to be unreliable underdrains. A "French drain" consists of a trench backfilled with rock. They are not to be used where a permanent solution is needed. Exceptions may be made for special cases such as where depth of the underdrain and soil conditions would conflict with industrial safety regulations. Under such circumstances a design that includes a filter fabric liner and permeable material backfill, without the perforated pipe may be used.

In addition to pipe underdrains, the following special purpose categories of subsurface drains are used to intercept, collect, and discharge groundwater.

- *Structural Section and Edge Drains*. Subsurface drainage systems that are primarily designed for the rapid removal of surface water infiltration from treated or untreated pavement structural section materials are called structural section drains or more typically edge drains. A 3-inch slotted plastic pipe with 3 rows of slots is the standard for structural section drains. Refer to Chapter 650, Pavement Drainage for more information on the drainage of the pavement structural section.

- *Horizontal Drains*. Horizontal drains are 1 1/2 inch perforated or slotted pipes placed in drilled holes bored into the aquifer or water bearing formations. They are installed in cut slopes and under fills more to guard against slides by relieving hydrostatic pressure than to prevent saturation of the roadbed. They may be used in varying lengths up to 1,000 feet on grades that range from 0 to 25 percent. A collection system to remove the intercepted water from the area is generally also required.

- *Prefabricated Geocomposite Drains*. Available in sheets or rolls, geocomposite drains provide a cost effective solution to subsurface drainage behind bridge abutments, wingwalls and retaining walls. Prefabricated subsurface drainage systems consist of a plastic drain core covered on one or both sides with a filter fabric.

- *Stabilization Trenches*. This category of subsurface drainage system is constructed in swales, ravines, and under sidehill fills to stabilize water logged fill foundations. The Geotechnical Design Report should contain depth and width of trench recommendations. Stabilization trenches may be only a few feet in width requiring a backhoe or similar type of excavation equipment, or they may be large enough for earth moving equipment such as dozers and scrapers to operate. Trenches wide enough to permit the use of earth moving equipment should be considered wherever feasible. A 1:1 side slope is commonly used.

  The excavated trench, including the side slopes, is covered with a thick blanket of permeable material. One or more perforated drain pipes, usually 8 inches to 12 inches in diameter, are placed at the bottom of the trench depending on the quantity of groundwater, type of material, and area to be stabilized.

  The alignment of the trench and collector pipe are often made parallel to the highway

CHAPTER 850
PHYSICAL STANDARDS

Topic 851 - General

Index 851.1 - Introduction

This chapter deals with the selection of drainage facility material type and sizes including pipes, pipe liners, pipe linings, drainage inlets and trench drains.

851.2 Selection of Material and Type

The choice of drainage facility material type and size is based on the following factors:

(1) *Physical and Structural Factors.* Of the many physical and structural considerations, some of the most important are:

(a) Durability.

(b) Headroom.

(c) Earth Loads.

(d) Bedding Conditions.

(e) Conduit Rigidity.

(f) Impact.

(g) Leak Resistance.

(2) *Hydraulic Factors.* Hydraulic considerations involve:

(a) Design Discharge.

(b) Shape, slope and cross sectional area of channel.

(c) Velocity of approach.

(d) Outlet velocity.

(e) Total available head.

(f) Bedload.

(g) Inlet and outlet conditions.

(h) Slope.

(i) Smoothness of conduit.

(j) Length.

Suggested values for Manning's Roughness coefficient (n) for design purposes are given in Table 851.2 for each type of conduit. See Index 864.3 for use of Manning's formula.

Topic 852 - Pipe Materials

852.1 Reinforced Concrete Pipe (RCP)

(1) *Durability.* RCP is generally precast prior to delivery to the project site. The durability of reinforced concrete pipe can be affected by abrasive flows or acids, chlorides and sulfate in the soil and water. See Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates.

The following measures increase the durability of reinforced concrete culverts:

(a) Cover Over Reinforcing Steel. Additional cover over the reinforcing steel should be specified where abrasion is likely to be severe as to appreciably shorten the design service life of a concrete culvert. This extra cover is also warranted under exposure to corrosive environments, see Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates. Extra cover over the reinforcing steel does not necessarily require extra wall thickness, as it may be possible to provide the additional cover and still obtain the specified D-load with standard wall thicknesses.

(b) Increase cement content.

(c) Reduce water content.

(d) Invert paving/plating.

(2) *Strength Requirements.*

(a) Design Standards. The strength of reinforced concrete pipe is determined by the load to produce a 0.01 inch crack under the "3-edge bearing test" called for in AASHTO Designations M 170, M 207M/M 207, and M 206M/M 206 for circular reinforced pipe, oval shaped reinforced pipe, and reinforced concrete pipe arches, respectively.

Table 851.2

Manning "n" Value for Alternative Pipe Materials(1)

Type of Conduit		Recommended Design Value	"n" Value Range
Corrugated Metal Pipe (2)			
(Annular and Helical) (3)			
2⅔" x ½"	corrugation	0.025	0.022 - 0.027
3" x 1"	"	0.028	0.027 - 0.028
5" x 1"	"	0.026	0.025 - 0.026
6" x 2"	"	0.035	0.033 - 0.035
9" x 2½"	"	0.035	0.033 - 0.037
Concrete Pipe			
Pre-cast		0.012	0.011 - 0.017
Cast-in-place		0.013	0.012 - 0.017
Concrete Box		0.013	0.012 - 0.018
Plastic Pipe (HDPE and PVC)			
Smooth Interior		0.012	0.010 - 0.013
Corrugated Interior		0.022	0.020 - 0.025
Spiral Rib Metal Pipe			
¾" (W) x 1" (D) @ 11½" o/c		0.013	0.011 - 0.015
¾" (W) x ¾" (D) @ 7½" o/c		0.013	0.012 - 0.015
¾" (W) x 1" (D) @ 8½" o/c		0.013	0.012 - 0.015
Composite Steel Spiral Rib Pipe		0.012	0.011 - 0.015
Steel Pipe, Ungalvanized		0.015	--
Cast Iron Pipe		0.015	--
Clay Sewer Pipe		0.013	--
Polymer Concrete Grated Line Drain		0.011	0.010 - 0.013

Notes:

(1) Tabulated n-values apply to circular pipes flowing full except for the grated line drain. See Note 5.

(2) For lined corrugated metal pipe, a composite roughness coefficient may be computed using the procedures outlined in the HDS No. 5, Hydraulic Design of Highway Culverts.

(3) Lower n-values may be possible for helical pipe under specific flow conditions (refer to FHWA's publication Hydraulic Flow Resistance Factors for Corrugated Metal Conduits), but in general, it is recommended that the tabulated n-value be used for both annular and helical corrugated pipes.

(4) For culverts operating under inlet control, barrel roughness does not impact the headwater. For culverts operating under outlet control barrel roughness is a significant factor. See Index 825.2 Culvert Flow.

(5) Grated Line Drain details are shown in Standard Plan D98C and described under Index 837.2(6) Grated Line Drains. This type of inlet can be used as an alternative at the locations described under Index 837.2(5) Slotted Drains. The carrying capacity is less than 18-inch slotted (pipe) drains.

852.6 Structural Metal Plate

(1) *Pipe and Arches.* Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 856.3M, N, O & P.

(2) *Strength Requirements.*

(a) Design Standards.

- Corrugation Profiles - Structural plate pipe and arches are available in a 6" x 2" corrugation for steel and a 9" x 2½" corrugation profile for aluminum.
- Metal Thickness - structural plate pipe and pipe arches are available in thickness as indicated on Tables 856.3M, N, O & P.
- Height of Fill - The allowable height of cover over structural plate pipe and pipe arches for the available diameters and thickness are shown on Tables 856.3M, N, O & P.

Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness.

(b) Basic Premise. To properly use the above mentioned tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe or arch size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

(c) Limitations. In using the tables, the following restrictions should be kept in mind.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. For steel pipe or pipe arches, where abrasive conditions are anticipated, additional metal thickness for the invert plate(s) or a paved invert should be provided when required to fulfill the design service life requirements. Table 855.2C may be used. See Index 855.2 Abrasion and Tables 855.2A, 855.2D and 855.2F.
- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Tables 856.3M & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a bearing pressure of 3 tons per square foot at the corners.

(d) Special Designs. If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for structural plate pipe or pipe arches are based, a special design prepared by DES - Structures Design is required.

(3) *Arches.* Design details with maximum allowable overfills for structural plate arches, with cast in place concrete footings may be obtained from DES - Structures Design.

(4) *Vehicular Underpasses.* Design details with maximum allowable overfills for structural plate vehicular underpasses with spans from 12 feet 2 inches to 20 feet 4 inches, inclusive, are given in the Standard Plans. These designs are based on bearing soil pressures from 1.4 tons per square foot to 5.8 tons per square foot.

(5) *Special Shapes.*

(a) Long Span. (Text Later)

- Arch

- Low Profile Arch
- High Profile Arch

(b) Ellipse. (Text Later)

- Vertical
- Horizontal

(6) *Tunnel Liner Plate.* The primary applications for tunnel liner plate include lining large structures in need of a structural repair, or culvert installations through an existing embankment that can be constructed by conventional tunnel methods. Typically, tunnel liner plate is not used for direct burial applications where structural metal plate pipe is recommended. DES - Structures Design will prepare designs upon request. See Index 853.7 for structural repairs.

852.7 Plastic Pipe

Plastic pipe is a generic term which currently includes two independent materials; the Standard Specifications states plastic pipe shall be made of either high density polyethylene (HDPE) or polyvinyl chloride (PVC) material. See Index 852.7(2)(a) Strength Requirements for allowed materials and wall profile types.

(1) *Durability.* Caltrans standards regarding the durability of plastic pipe are based on the long term performance of its material properties. Both forms of plastic pipe culverts (HDPE and PVC) exhibit good abrasion resistance and are virtually corrosion free. See Index 855.2 Abrasion and Index 855.5 Fire. Also, see Tables 855.2A, 855.2E and 855.2F. The primary environmental factor currently considered in limiting service life of plastic materials is ultraviolet (UV) radiation, typically from sunlight exposure. While virtually all plastic pipes contain some amount of UV protection, the level of protection is not equal. Polyvinyl chloride resins used for pipe rarely incorporate UV protection (typically Titanium Dioxide) in amounts adequate to offset long term exposure to direct sunlight. Therefore, frequent exposure (e.g., cross culverts with exposed ends) can lead to brittleness and such situations should be avoided. Conversely, testing performed to date on HDPE products conforming to specification requirements for inclusion of carbon black have exhibited adequate UV resistance. PVC pipe exposed to freezing conditions can also experience brittleness and such situations should be avoided if there is potential for impact loadings, such as maintenance equipment or heavy (3" or larger) bedload during periods of freeze. Plastic pipes can also fail from long term stress that leads to crack growth and from chemical degradation. Improvements in plastic resin specifications and testing requirements has led to increased resistance to slow crack growth. Inclusion of anti-oxidants in the material formulation is the most common form of delaying the onset of chemical degradation, but more thorough testing and assessment protocols need to be developed to more accurately estimate long term performance characteristics and durability.

(2) *Strength Requirements.*

(a) Design Standards

- Materials - Plastic pipe shall be either Type C (corrugated exterior and interior) corrugated polyethylene pipe, Type S (corrugated exterior and smooth interior) corrugated polyethylene pipe, ribbed profile wall polyethylene pipe, corrugated polyvinyl chloride pipe, or ribbed polyvinyl chloride pipe.
- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5.

852.8 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements, or for certain jacked pipe operations (e.g., auger boring).

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

CHAPTER 870 CHANNEL AND SHORE PROTECTION - EROSION CONTROL

Topic 871 - General

Index 871.1 - Introduction

Highways are often attracted to parallel locations along streams, coastal zones and lake shores. These locations are under attack from the action of waves and flowing water that may require protective measures.

Channel and shore protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water and wave action on highway facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control.

Refer to Topic 874 for definition of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802).

There are a number of ways to deal with the problem of wave action and stream flow.

- The simplest way and generally the surest of success and permanence, is to locate the roadway away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the roadway to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the embankment with a more resistant material like rock slope protection. The type of material to be used for the protection is discussed under Topic 872.
- A third method is to reduce the force of the attacking water. This is often done by means of retards, permeable jetties and various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.
- Another method is to direct the attacking water away from the embankment. In the case of wave attack, additional beach may be created between the embankment and the water by means of groins and sills which trap littoral drift or hold imported sand. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of jetties, baffles, deflectors, groins or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to highways.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness. Retards, baffles and jetties can simulate the effect of trees and boulders along natural banks and in overflow channels.

- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.
- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.
- *Expendability*. The primary objectives of the design are the safety of the traveling public and the security of the highway, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.
- *Dependability*. An expensive structure is warranted primarily where highways carry high traffic volumes, where no detour is available, or where roadway replacement is very expensive.
- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.
- *Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in shore protection and the specified minimum should not be lowered without increasing the mass of stones. For example, 10 percent decrease in specific gravity requires a 55 percent increase in mass (say from a 9 ton stone to a 14 ton stone) for equivalent protection.
- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.
- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

871.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to slope protection measures are repeated here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following five circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 11, Design of Riprap Revetment (2000)
- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2000)
- HEC 15, Design of Roadside Channels with Flexible Linings (2000).
- HEC 18, Evaluating Scour at Bridges (2001)
- HEC 20, Stream Stability at Highway Structures (1995)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2001)
- HEC 25, Tidal Hydrology, Hydraulics, and Scour at Bridges (2004)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) -- A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction, and Volume XI - Guidelines for Highways Along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (MDM) (2003) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection; and 18 – Coastal Zone. The MDM provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) U.S. Army Corps of Engineers Manuals. The following manuals are used throughout the U.S. as a primary resource for the design and analysis of coastal features:

- Shore Protection Manual (SPM) (1984) – Comprehensive two volume guidance on wave and shore processes and methods for shore protection. No longer in publication but still referenced pending completion of the Coastal Engineering Manual.
- Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).
- Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents, once complete, will supersede the SPM and EM 1110-2-1614. As of this writing Parts I thru V and the appendix are completed and available. Parts V and VI are considered "Engineering – Based" and present information on design process and selection of appropriate types of solutions to various coastal challenges.

Topic 872 - Planning and Location Studies

872.1 Planning

The development of cost effective protective works requires careful planning. Planning begins with site investigation. The selection of the class of protection can be determined during or following site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable. See the FHWA's HDS No. 6, River Engineering for Highway Encroachments for a complete and thorough discussion of hydraulic and environmental design considerations associated with hydraulic structures in moveable boundary waterways.

Some specific site conditions that may dictate selection of a class and type of protection different from those shown in Table 872.1 are:

- Available right of way.
- Available materials.
- Possible damage to other properties through streamflow diversion or increased velocity.
- Environmental concerns.
- Channel capacity or conveyance.
- Conformance to new or existing structures.
- Provisions for side drainage, either surface waters or intersecting streams or rivers.

The first step is to determine the limits of the protection with respect to length, depth and the degree of security required.

Considerations at this stage are:

- The severity of attack.
- The present alignment of the stream or river and potential meander changes.
- The ratio of cost of highway replacement versus cost of protection.
- Whether the protection need be permanent or temporary.
- Analysis of foundation and materials explorations.

The second step is the selection and layout of protective elements in relation to the highway facility.

872.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device. For additional information on specific material types and shapes see Topic 873, Design Concepts.

There are two basic classes of protection, armor treatment and training works. Table 872.1 relates different location environments to these classes of protection.

872.3 Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection are affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) Young Valley. Typically young valleys are narrow V-shaped valleys with streams on steep gradients. At flood stage, the stream flow covers all or most of the valley floor. The usual situation for such locations is a structure crossing a well-defined channel in which the design discharge will flow at a moderate to high velocity.

(a) Cross-Channel Location. A cross channel location is a highway crossing a stream on normal or skewed alignment. The erosive forces of parallel flow associated with a normal crossing are generally less of a threat than the impinging and eddy flows associated with a skewed crossing. The effect of constriction by projection of the roadway embankment into the channel should be assessed.

Characteristics to be considered include:

- Stream velocity.
- Scouring action of stream.
- Bank stability.
- Channel constrictions (artificial or natural).
- Nature of flow (tangential or curvilinear).
- Areas of impingement at various stages.
- Security of leading and trailing edges.

Common protection failures occur from:

- Undermining of the toe (inadequate depth/size of foundation), see Figure 872.1 and Table 872.2.
- Local erosion due to eddy currents.
- Inadequate upstream and downstream terminals or transitions to erosion-resistant banks or outcrops.
- Structural inadequacy at points of impingement overtopping.
- Inadequate rock size, see Table 872.2.
- Lack of proper gradation/ layering/ RSP fabric, leading to loss of embankment, see Table 872.2.

Figure 872.1

Slope Failure Due to Loss of Toe

Table 872.1

Guide to Selection of Protection

Location	Armor										Training										
	Flexible				Rigid						Guide Dikes Retards & Jetties				Groins				Baffles		
			Mattresses																		
	Vegetation	Riprap	Gabions	Conc. Blocks	Fabric Filled	Grouted Rock	Stacked Conc.	Conc. Lined	Cribs	Bulk heads	Earth	Fencing	Piling	Other	Rock	Grouted Rock	Piling	Other	Drop Structure	Fencing	Rock Earth
Cross Channel																					
Young Valley		X	Ø			X			X	X											
Mature Valley		X	Ø	Ø	Ø	X			X	X	X	X	X	X	X	X			X	X	X
Parallel Encroachment																					
Young Valley		X	Ø			X			X	X											
Mature Valley	X	X	Ø	Ø	Ø	X	Ø		X	X	X	X	X	X	X	X	X	X	X	X	X
Lakes and Tidals Basins	X	X	Ø	Ø	Ø	X	Ø			X											
Ocean Front		X	Ø	Ø						X					X	X	X	X			
Desert-wash																					
Top debris cone		X	Ø	Ø	Ø	X				X											
Center debris cone		X	Ø	Ø	Ø	X													X	X	X
Bottom debris cone		X	Ø	Ø	Ø	X													X	X	X
Overflow and floodplain	X	X	Ø			X	Ø				X	X	X	X							
Artificial channel	X	X	Ø	Ø	Ø	X	Ø	X													
Culvert																					
Inlet		X				X	Ø			X											
Outlet		X				X	Ø			X											
Bridge																					
Abutment		X		Ø	Ø	X	Ø	X													
Upstream		X				X					X	X	X	X							
Downstream		X				X					X	X	X	X					X		
Roadside ditch	X	X				X	Ø	X													

Ø Where large rock for riprap is not available

Table 872.2

Failure Modes and Effects Analysis for Riprap Revetment

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slope or slump (slope failure)	Disruption of armor layer	Catastrophic failure	• Mound of rock at bank toe • Unprotected upper bank	• Reduce bank slope • Use more angular or smaller rock • Use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	• Rock moved downstream from original location • Exposure of filter	• Increase rock size • Modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	• Scalloping of upper bank • Bank cutting • Void beneath and between rocks	• Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption or armor layer	Catastrophic failure	• Slumping of rock • Unprotected upper bank	• Increase size, thickness, depth or extent of toe or key

Any of the more substantial armor treatments can function properly in such exposures providing precautions are taken to alleviate the probable causes of failure. If the foundation is questionable for concreted-rock or other rigid types it would not be necessary to reject them from consideration but only to provide a more acceptable treatment of the foundation, such as heavy rock or sheet piling.

Whether the highway crosses a stream channel on a bridge or over a culvert, economic considerations often lead to constriction of the waterway. The most common constriction is in width, to shorten the structure. Next in frequency is obstruction by piers and bents of bridges or partitions of multiple culverts.

The risk of constricting the width of the waterway is closely related to the relative conveyance of the natural waterway obstructed, the channel scour, and to the channel migration. Constricting the width of flow at structures has the following effects:

- Increase in the upstream water surface elevation (backwater profile).
- Increase in flow velocity through the structure opening (waterway).
- Causes eddy currents around the upstream and downstream ends of the structure.

Unless protection is provided the eddy currents can erode the approach roadway embankment and the accelerated flow can cause scour at bridge abutments. The effects of erosion can be reduced by providing transitions from natural to constricted and back to natural sections, either by relatively short wingwalls or by relatively long training embankments or structures.

Channel changes, if properly designed, can improve conditions of a crossing by reducing skew and curvature and enlarging the main channel. Unfortunately there are "side effects" which actually increase erosion potential. Velocity is almost always increased by the channel change, both by a reduction of channel roughness and increase of slope due to channel shortening. In addition, channel changes affecting stream gradient may have upstream and/or downstream effects as the stream adjusts in relation to its sediment load.

At crossing locations, lateral erosion can be controlled by positive protection, such as armor on the banks, rock spurs to deflect currents away from the banks, retards to reduce riparian velocity, or vertical walls or bulkheads. The life cycle cost of such devices should be considered in the economic studies to choose a bridge length which minimizes total cost.

Accurate estimates of anticipated scour depths are a prerequisite for safe, cost effective designs. Design criteria require that bridge foundations be placed below anticipated scour depths. For this reason the design of protection to control scour at such locations is seldom necessary for new construction. However, if scour may undercut the toes of dikes or embankments positive methods including self-adjusting armor at the toe, jetties or retards to divert scouring currents away from the toe, or sill-shaped baffles interrupting transport of bedloads should be considered.

There is the potential for instability from saturated or inundated embankments at crossings with embankments projecting into the channel. Failures are usually reported as "washouts", but several distinct processes should be noted:

- Saturation of an embankment reduces its angle of repose. Granular fills with high permeability may "dissolve" steadily or slough progressively. Cohesive fills are less permeable, but failures have occurred during falling stages.
- As eddies carve scallops in the embankment, saturation can be accelerated and complete failure may be rapid. Partial or total losses can occur

due to an upstream eddy, a downstream eddy, or both eddies eroding toward a central conjunction. Training devices or armor can be employed to prevent damage.

- If the fill is pervious and the pavement overtopped, the buoyant pressure under the slab will exceed the weight of slab and shallow overflow by the pressure head of the hydraulic drop at the shoulder line. A flat slab of thickness, t, will float when the upstream stage is 4t higher than the top of the slab. Thereafter the saturated fill usually fails rapidly by a combination of erosion and sloughing. This problem can occur or be increased when curbs, dikes, or emergency sandbags maintain a differential stage at the embankment shoulder. It is increased by an impervious or less pervious mass within the fill. Control of flotation, insofar as bank protection is concerned, should be obtained by using impervious armor on the upstream face of the embankment and a pervious armor on the downstream face.

Culvert problem locations generally occur in and along the downstream transition. Sharp divergence of the high velocity flow develops outward components of velocity which attack the banks directly by impingement and indirectly by eddies entrained in quieter water. Downward components and the high velocity near the bed cause the scour at the end of the apron.

Standard plans of warped wingwalls have been developed for a smooth transition from the culvert to a trapezoidal channel section. A rough revetment extension to the concrete wingwalls is often necessary to reduce high velocity to approximate natural flow. Energy dissipaters may be used to shorten the deceleration process when such a transition would be too long to be economical. Bank protection at the end of wingwalls is more cost effective in most cases.

(b) Parallel Location. With parallel locations the risk of erosion damage along young streams increases where valleys narrow and gradients steepen. The risk of erosion damage is greatest along the outer bend of natural meanders or where highway embankment encroaches on the main channel.

The *encroaching* parallel location is very common, especially for highways following mountain streams in narrow young valleys or canyons. Much of the roadway is supported on top of the bank or a berm and the outer embankment encroaches on the channel in a zone of low to moderate velocity. Channel banks are generally stable and protection, except at points of impingement, is seldom necessary.

The *constricting* parallel location is an extreme case of encroaching location, causing such impairment of channel that acceleration of the stream through the constriction increases its attack on the highway embankment requiring extra protection, or additional waterway must be provided by deepening or widening along the far bank of the stream.

In young valleys, streams are capable of high velocity flows during flood stages that may be damaging to adjacent highway facilities. Locating the highway to higher ground or solid support is always the preferred alternative when practical.

Characteristics to be considered include:

- High velocity flow.
- Narrow confined channels.
- Accentuated impingement.
- Swift overflow.
- Disturbed flow due to rock outcrops on the banks or within the main channel.
- Alterations in flow patterns due to the entrance of side streams into the main channel.

Protective methods that have proven effective are:

- Rock slope protection.
- Concreted-rock slope protection.
- Walls of masonry and concrete.
- Articulated concrete block revetments.
- Sacked concrete.
- Cribs walls of various materials.

(2) *Mature Valley*. Typically mature valleys are broad V-shaped valleys with associated flood plains. The gradient and velocity of the stream are low to moderate. In addition to the general information previously given, the following applies to mature valleys.

(a) Cross-Channel Location. The usual situation is a structure crossing a braided or meandering normal flow channel. The marginal area subject to overflow is usually traversed by the highway on a raised embankment and may have long approaches extending from both banks.

Characteristics to be considered include:

- Shifting of the main channel.
- Skew of the stream to the structure.
- Foundation in deep alluvium.
- Erodible embankment materials.
- Channel constrictions, either artificial or natural, which may affect or control the future course of the stream.
- Variable flow characteristics at various stages.
- Stream acceleration at the structure.

Armor protection has proven effective to prevent erosion of road approach embankments, supplemented if necessary by stream training devices such as guide dikes, permeable retards or jetties to direct the stream through the structure. The abutments should not depend on the training dikes to protect them from erosion and scour. At bridge ends one of the more substantial armor types may be required, but bridge approach embankments affected only by overflow seldom require more than a light revetment, such as a thin layer of rocky material, vegetation, or a fencing along the toe of slope. For channel flow control upstream, the size and type of training system ranges from pile wings for high velocity, through permeable jetties for moderate velocity, to the earth dike suitable for low velocity.

The more common failures in this situation occur from:

- Lack of upstream control of channel alignment.
- Damage of unprotected embankments by overflow and return flow.
- Undercut foundations.
- Formation of eddies at abrupt changes in channel.
- Stranding of drift in the converging channel.

(b) Parallel Location. Parallel highways along mature rivers are often situated on or behind levees built, protected and maintained by other agencies. Along other streams, rather extensive protective measures may be required to control the action of these meandering streams.

Channel change is an important factor in locations parallel to mature streams. The channel change may be to close an embayment, to cut off an oxbow, or to shift the alignment of a long reach of a stream. In any case, positive means must be adopted to prevent the return of the stream to its natural course. For a straight channel, the upstream end is critical, usually requiring bank protection equivalent to the facing of a dam. On a curved channel change, all of the outer bend may be critical, requiring continuous protection. For a channel much shorter than the natural channel, particularly for elimination of an oxbow, the corresponding increase in gradient may require transverse weirs as grade control structures to prevent undercutting. For unusual channel changes, preliminary plans and hydraulic data must be submitted to FHWA for approval (see Index 805.5).

(3) *Lakes and Tidal Basins.* Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of waters generate waves. Height of waves is a function of fetch and depth. Erosion along embankments behind shallow coves is reduced because the higher waves break upon reaching a shoal in shallow water. The threat of erosion in deep water at headlands or along causeways is increased. Constant exposure to even the rippling of tiny waves may cause severe erosion of some soils.

Older lakes normally have thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit the available foundation. It is usually more practical to use lightweight or self-adjusting armor types supported by the soft bed materials than to excavate the mud to stiffer underlying soils.

In fresh waters, effective protection can often be provided by the establishment of vegetation, but planners should not overlook the possibility of moderate erosion before the vegetative cover becomes established. A light armor treatment should be adequate for this transitional period.

(4) *Ocean Front Locations.* Wave action is the erosive force affecting the reliability of highway locations along the coast. The corrosive effect of salt water is also a major concern for hydraulic structures located along the coastline. Headlands and rocks that have historically withstood the relentless pounding of tide and waves can usually be relied on to continue to protect adjacent highway locations founded upon them. The need for shore protection structures is, therefore, generally limited to highway locations along the top or bottom of bluffs having a history of sloughing and along beach fronts.

Beach protection considerations include:

- Attack by waves.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- Foundation for protective structures.

Wave attack on a beach is less severe than on a headland, due to the gradual shoaling of the bed which trips incoming waves into a series of breakers called a surf.

Littoral drift of beach sands may either be an asset or a liability. If sand is plentiful, a new beach will be built in front of the highway embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If sand supply is less plentiful or subject to seasonal variations, the new beach can be induced or retained by groins.

If sand is in scant supply, backwash from a revetment tends to degrade the beach or bed even more than the seasonal variation, and an allowance should be made for this scour when designing the revetment, both as to weight of stones and depth of foundation. Groins may be ineffective for such locations; if they succeeded in trapping some littoral drift, downcoast beaches would recede from undernourishment.

Seasonal shifts of the shore line result from combinations of:

- Ranges of tide.
- Reversal of littoral currents.
- Changed direction of prevailing onshore winds.
- Attack by swell.

Generally the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may result in deposition at the other. Observations made during location assessment should include investigation of this phenomenon. For strands, the hazard may be avoided by locating the highway on the backshore facing the lagoon.

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantle of sand. Bed stones and even gravity walls have been founded successfully on such foundations. Spits and strands, however, are radically different, often with softer clays or organic materials underlying the sand. Sand is usually plentiful at such locations, subsidence is a greater hazard than scour, and location should anticipate a "floating" foundation for flexible, self-adjusting types of protection.

In planning ocean-front locations, the primary decision is a choice of (1) alignment far enough inshore to avoid wave attack, (2) armor on the embankment face, or (3) off shore devices like groins to aggrade the beach at embankment toe.

See Index 873.3(2) for further discussion on determining the size of rocks necessary in shore protection for various wave heights.

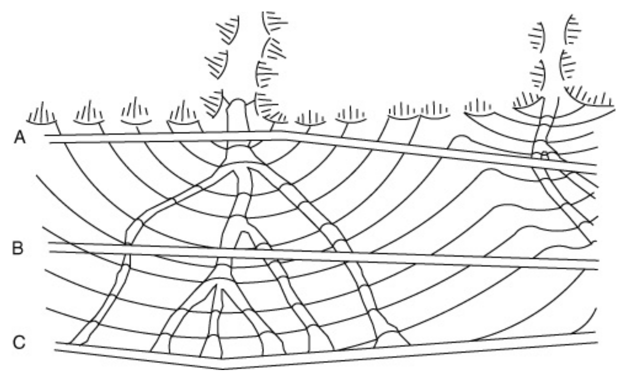
(5) *Desert Wash Locations.* Special consideration should be given to highway locations across the natural geographical features of desert washes, sand dunes, and other similar regions.

Desert washes are a prominent feature of the physiography of California. Many long stretches of highway are located across a succession of outwash cones. Infrequent discharge is typically wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridge the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

An important consideration at these locations is instability of the channel, see Figure 872.2. For a location at the top of a cone (Line A), discharge is maximum, but the single channel emerging from the uplands is usually stable. For a location at the bottom of the cone (Line C), instability is maximum with poor definition of the channel, but discharge is reduced by infiltration and stream dispersion. The energy of the stream is usually dissipated so that any protection required is minimal. The least desirable location is midway between top and bottom (Line B), where large discharge may approach the highway in any of several old channels or break out on a new line. Control may require dikes continuously from the top of the cone to such a mid-cone site with slope protection added near the highway where the converging flow is accelerated. See Figure 872.3, which depicts a typical alluvial fan.

Figure 872.2

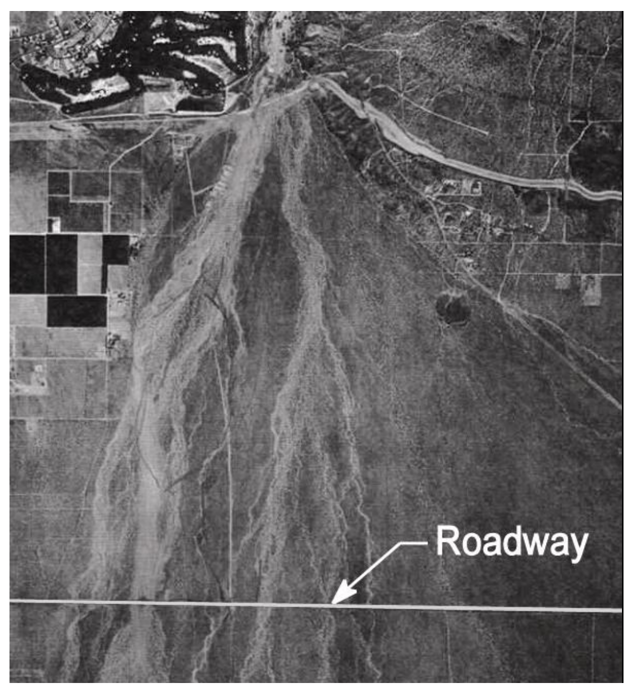
Alternative Highway Locations Across Debris Cone



(A) crosses at a single definite channel,
(B) a series of unstable indefinite channels and
(C) a widely dispersed and diminished flow.

Figure 872.3

Alluvial Fan



Typical multi-channel stream threads on alluvial fan. Note location of roadway crossing unstable channels.

Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.4. Re-alignment to a stable location should be the first consideration, but restrictions imposed by federal or state agencies (National Park Service, USDA Forest Service, etc.) may preclude that option, somewhat similar to transverse crossings. The designer may need to consider allowing frequent overtopping and increased sediment removal maintenance since an "all weather design" within these regimes can often lead to large scale roadway washout.

Figure 872.4

Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

Characteristics to be considered include:

- The intensity of rainfall and subsequent run-off.
- The relatively large volumes of solids that are carried in such run-off.
- The lack of definition and permanence of the channel.
- The scour depths that can be anticipated.
- The lack of good foundation.

Effective protective methods include armor along the highway and at structures and the probable need for baffles to control the direction and velocity of flow. Installations of rock, fence, palisades, slope paving, and dikes have been successful.

The Federal Emergency Management Agency (FEMA) Flood Hazard Mapping website contains information on recognizing alluvial fan landforms and methods for defining active and inactive areas. See their "Guidelines for Determining Flood Hazards on Alluvial Fans" at http://www.fema.gov/fhm/ft_tocs.shtm.

872.4 Data Needs

The types and amount of data needed for planning and analysis of bank and shore protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. The data that is collected and developed including preliminary calculations, and alternatives considered should be documented in project development reports (Environmental Document, Project Report, etc.) or as a minimum in the project file. These records serve to guide the detailed designs, and provide reference background for analysis of environmental impacts and other needs such as permit applications and historical documentation for any litigation which may arise.

Recommendations for data needs can be requested from the District Hydraulics Engineer or determined from Chapter 8 of FHWA's HDS No. 6, for a more complete discussion of data needs for highway crossings and encroachments on rivers. Further references to data needs are contained in Chapter 810, Hydrology and FHWA's HDS No. 2, Highway Hydrology.

Topic 873 - Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economical analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of State Highway Drainage Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures. The Caltrans Bank and Shore Protection Committee is available on request to provide expert advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: concreted-rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Lower cost replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone may be available in roadway excavation.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency.

A suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks.

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) *Streambank Locations.* The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter 3, HDS No. 6, and to Chapter 4, HDS No. 6, for further information on sediment transport.

(2) *Ocean & Lake Shore Locations.* Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(a) Design High Water Level. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owners schedule of operation.

Except for inland tidal basins affected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at MLLW. To convert this to MSL datum there must be subtracted a datum equation (2.5 feet to 3.9 feet) factor. If datum differs from MSL datum, a further correction is necessary. These steps should be undertaken with care and independently checked. Common errors are:

- Ignoring the datum equation.
- Adding the factor instead of subtracting it.
- Using half the diurnal range as the stage of high water.

To clarify the determination of design high-water, Fig. 873.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is MHHW, and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

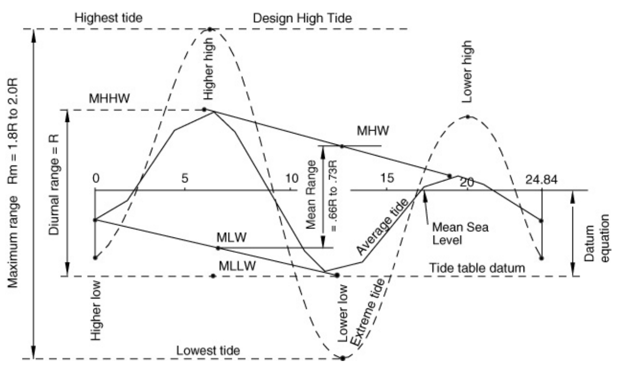
Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained.

Figure 873.2A

Nomenclature of Tidal Ranges



Because of the great variation of tidal elements, Figure 873.2A was not drawn to scale.

The elevation of the design high tide may be taken as mean sea level (MSL) plus one-half the maximum tidal range (Rm).

(b) Design Wave Heights.

(1) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. It is recommended that for ocean shore protection designs the assistance of the U.S. Army Corp of Engineers be requested.

Shore protection structures are generally designed to withstand the wave that induces the highest forces on the structure over its economic service life. The design wave is analogous to the design storm considerations for determining return frequency. A starting point of reference for shore protection design is the maximum significant wave height that can occur once in about 20-years. Economic and risk considerations involved in selecting the design wave for a specific project are basically the same as those used in the analysis of other highway drainage structures.

(2) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual – Part II, for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from boat generated waves must be estimated from observations.

The surface of any large body of water will contain many waves differing in height, period, and direction of propagation. A representative wave height used in the design of bank and shore protection is the significant wave height, H_s. The significant wave height is the average height of the highest one-third of all the waves in a wave train for the time interval (return frequency) under consideration. Thus, the design wave height generally used is the significant wave height, H_s, for a 20-year return period.

Other design wave heights can also be designated, such as H_{10} and H_1. The H_{10} design wave is the average of the highest 10 percent of all waves, and the H_1 design wave is the average of the highest 1 percent of all waves. The relationship of H_{10} and H_1 to H_s can be approximated as follows:

$H_{10} = 1.27\ H_s$ and $H_1 = 1.67\ H_s$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

(3) Wave Characteristics. Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gage records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

(4) Predicting Wind Generated Waves. The height of wind generated waves is a function of fetch length, windspeed, wind duration, and the depth of the water.

(a) Hindcasting -- The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods. Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

(b) Forecasting -- Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs.

The following assumptions pertain to these simplified methods:

- The fetch is short, 75 miles or less
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameters should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional or shallow water rather than in deep water.

The height of wind generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, wind-speed, and fetch length.

Procedures for predicting wind generated waves are complex and our understanding and ability to describe wave phenomena, especially in the region of the coastal zone, is limited. Many aspects of physics and fluid mechanics of wave energy have only minor influence on the design of shore protection for highway purposes. Designers interested in a more complete discussion on the rudiments of wave mechanics should consult the U.S.Army Corps of Engineers' Coastal Engineering Manual – Part II.

An initial estimate of wind generated significant wave heights can be made by using Figure 873.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. It is recommended that advice from the Army Corps of Engineers be obtained to refine significant wave heights, H_S, greater than 2 feet.

(5) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height.

The relationship of the maximum height of breaker which will expend its energy upon the protection, H_b, and the depth of water at the slope protection, d_s, which the wave must pass over are illustrated in Figure 873.2C.

The following diagram, with some specific references to the SPM, summarizes an overly simplified procedure that may be used for highway purposes to estimate wind generated waves and establish a design wave height for shore protection.

(6) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by policy for projects subject to California Coastal Commission (CCC) jurisdiction (see CCC guidance document "Beach Erosion and Response," December 1999). Procedures for estimating wave run-up for rough surfaces (e.g., RSP) are contained in the U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads, (EM 1110-2-1614) published in 1995.

Determining Design Wave

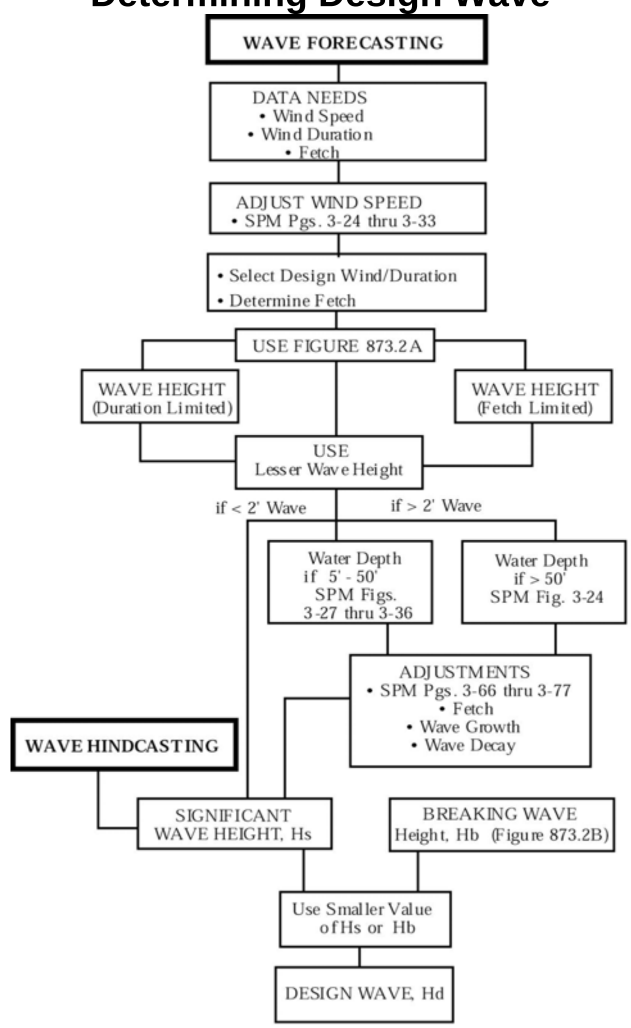


Figure 873.2B

Significant Wave Height Prediction Nomograph

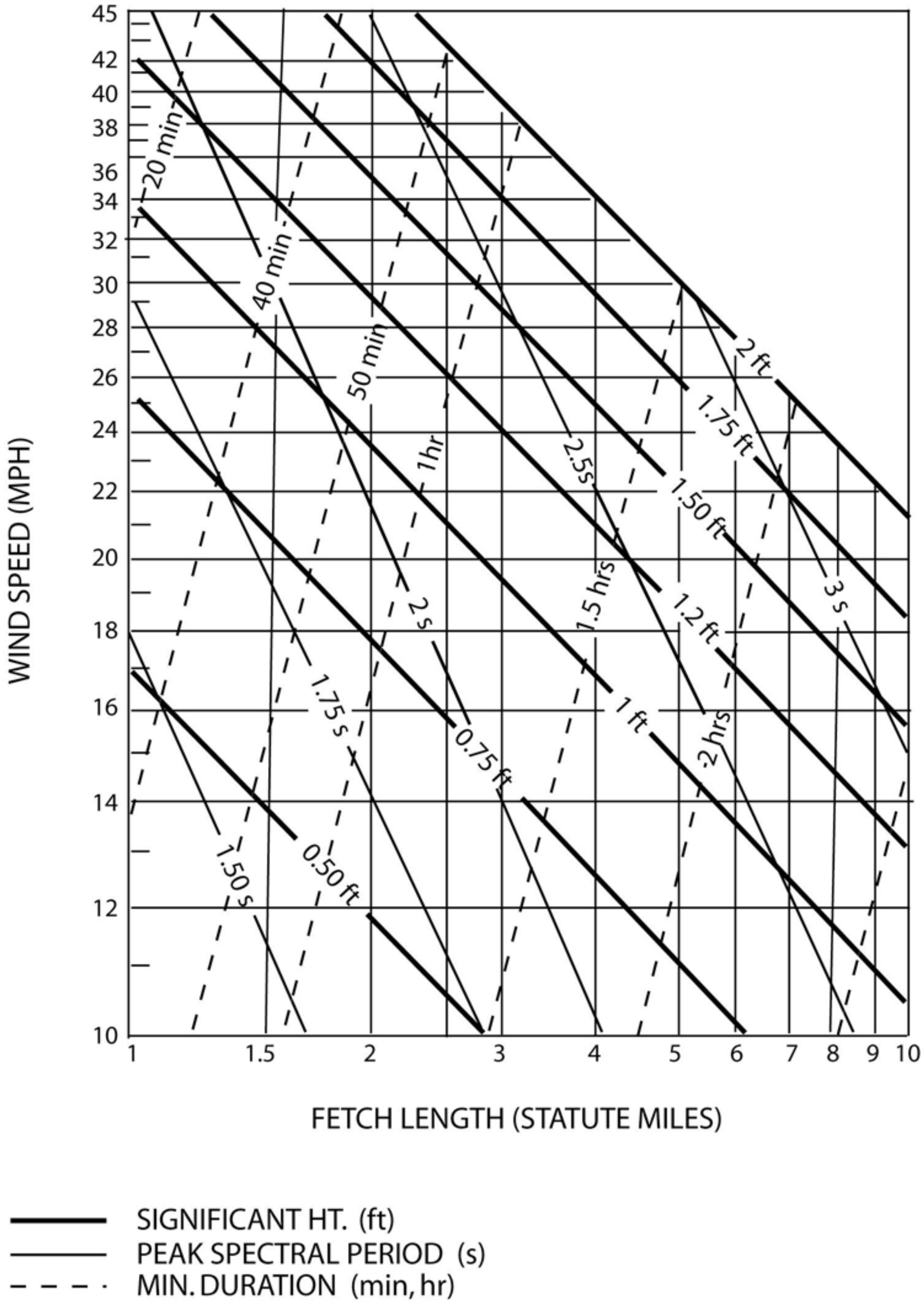
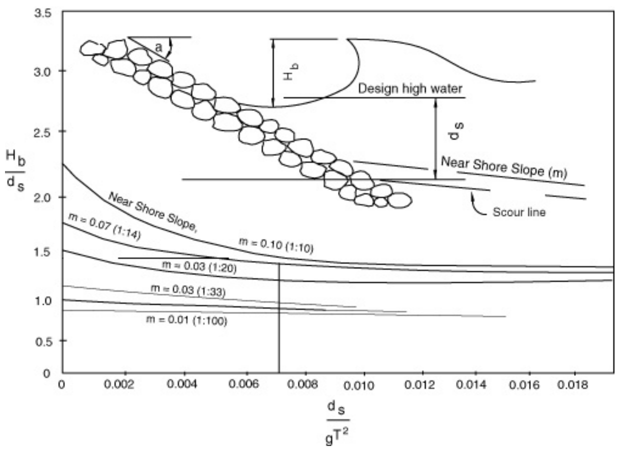


Figure 873.2C
Design Breaker Wave



Example
By using hindcast methods, the significant wave height (H_s) has been estimated at 4 feet with a 3 second period. Find the design wave height (H_d) for the slope protection if the depth of water (d) is only 2 feet and the nearshore slope (m) is 1:10.

Solution

$$\frac{d_s}{gT^2} = \frac{2 \text{ ft}}{(32.2 \text{ ft/s}^2) \times (3 \text{ sec})^2} = 0.007$$

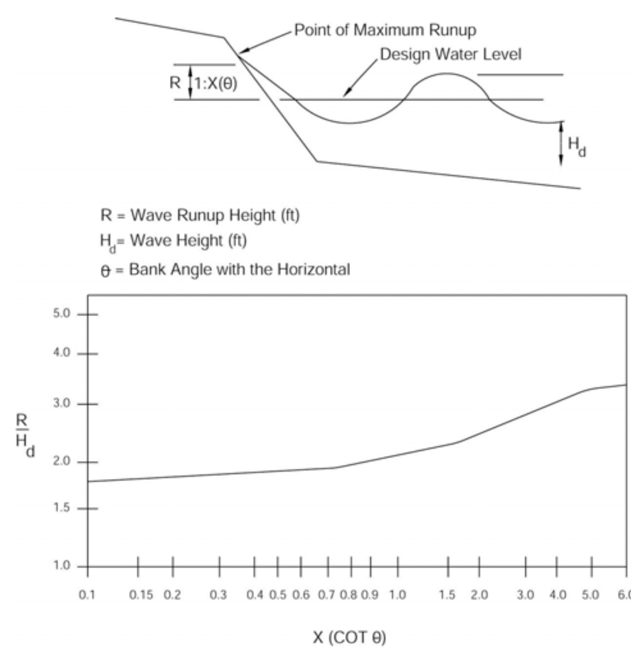
From Graph) - $H_b/d_s = 1.4$
$H_b = 2 \times 1.4 = 2.8$ ft

Answer
Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 2.8 feet.
T = Wave Period (SPM)

Procedures for estimating wave run-up for smooth surfaces (e.g., concrete paved slopes) and for vertical and curved face walls are contained in the U.S. Army Corps of Engineers, Shore Protection Manual, 1984. See Figure 873.2D for estimating wave run-up on smooth slopes for wave heights of 2 feet or less.

In protected bays and estuaries, waves generated by recreational or commercial boat traffic and other watercraft may dominate the design over wind generated waves. Direct observation and measurements during high tidal cycles may provide the designer the most useful tool for establishing wave run-up for these situations.

Figure 873.2D
Wave Run-up on Smooth Impermeable Slope



(c) Littoral Processes. Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can effect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of

Engineers' Coastal Engineering Manual (CEM) – Part III.

873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self adjusting) or rigid.

Hard armoring of stream banks and shorelines, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared. The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, "California Bank and Shore Rock Slope Protection Design". Within that report, the methodology for RSP design adopted as the Departmental standard, is the California Bank and Shore, (CABS), layered design. The full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

This design method, which is applied with slight variation to ocean and lake shores vs. stream banks, and is also followed for concreted RSP designs, is the only protection method as of this writing that has been formally adopted by the Caltrans Bank and Shore Protection Committee. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP's) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Other armor types listed below and described throughout this Chapter are viable and may be used, upon approval of the Headquarters Hydraulic Engineer or Caltrans Bank and Shore Protection Committee, where conditions warrant. Although the additional step of headquarters approval of these non-standard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The Office of Highway Drainage Design is coordinating with the California Department of Fish and Game in assessing vegetative and bioengineered armoring methods for possible adoption into the Departments' standards. The Headquarters Hydraulic Engineer can provide information on the status of that effort upon request. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Broken concrete slope protection.
- Broken concrete, uncoursed.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.
- Rock filled cellular mats.

(b) Rigid Types.

- Concreted-rock slope protection.
- Sacked concrete slope protection.
- Concrete slope protection.
- Concrete filled fabric slope protection.
- Air-blown mortar.
- Soil cement slope protection.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a

drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks with open cells
- Brush layering
- Rock riprap in sizes smaller than backing No. 3

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls
- Sheet piling
- Sea Walls

(d) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be

influenced or dictated by economics, or the feasibility of handling heavy individual units.

(2) *Flexible Revetments.*

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of very large rock may require large cranes or equipment with special lifting capabilities).
- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness.
- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile and backing material.
- Face slope.

(a) Placement -- Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A by 25 percent.

(b) Foundation Treatment -- The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in the CHANLPRO Program developed by the U.S. Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: http://chl.erdc.usace.army.mil/CHL.aspx?p=s&a=Software;3 along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) Embankment Considerations -- Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) Rock Slope Protection Fabric and Inner Layers of Rock -- The layered method of designing RSP installations was developed prior to widespread availability of the geosynthetic fabrics which are described in Standard Specification Section 88 – 1.04. The RSP fabric and multiple layers of rock ensure that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing backing No. 3 or similar small, well graded materials. See Index 873.3(2)(a)(1)(e) "Gravel Filter." Under special circumstances, the designer may consider allowing holes to be cut in the RSP fabric, generally to facilitate more rapid/extensive rooting of woody vegetation through the RSP revetment. This practice is only necessary for deeply rooted plant species. Holes in RSP fabric should not be cut below the stage of the 2-year return period event. The District Hydraulic Unit should be consulted for advice prior to any determination to cut or otherwise modify standard installation of RSP fabric.

Additionally, stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of the layered design is not feasible and large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) Gravel Filter -- Generally RSP fabric should always be used unless there is a permit requirement for establishment of vegetation that precludes the placement of fabric due to inadequate root penetration. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Game and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter.

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes (see Figure 873.3A) use lighter stones in

a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.

- Use stone of adequate weight to resist erosion, derived from Figure 873.3A.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed in two or more layers of rock sizes, with progressively smaller rock toward native bank to prohibit loss of soil fines.
- Be constructed of rock of such shape as to form a stable protection structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V

(a) Stone Size -- Where stream velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A.

The nomograph is derived from the following formula:

$$W = \frac{0.00002V^6 sg_r \csc^3(\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

sg_r = specific gravity of stones

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

V = 2/3 average stream velocity, fps (flow parallel to bank) or 4/3 average stream velocity, fps (flow impinging on bank)

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection.

(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in

Figure 873.3A

Nomograph of Stream-Bank Rock Slope Protection

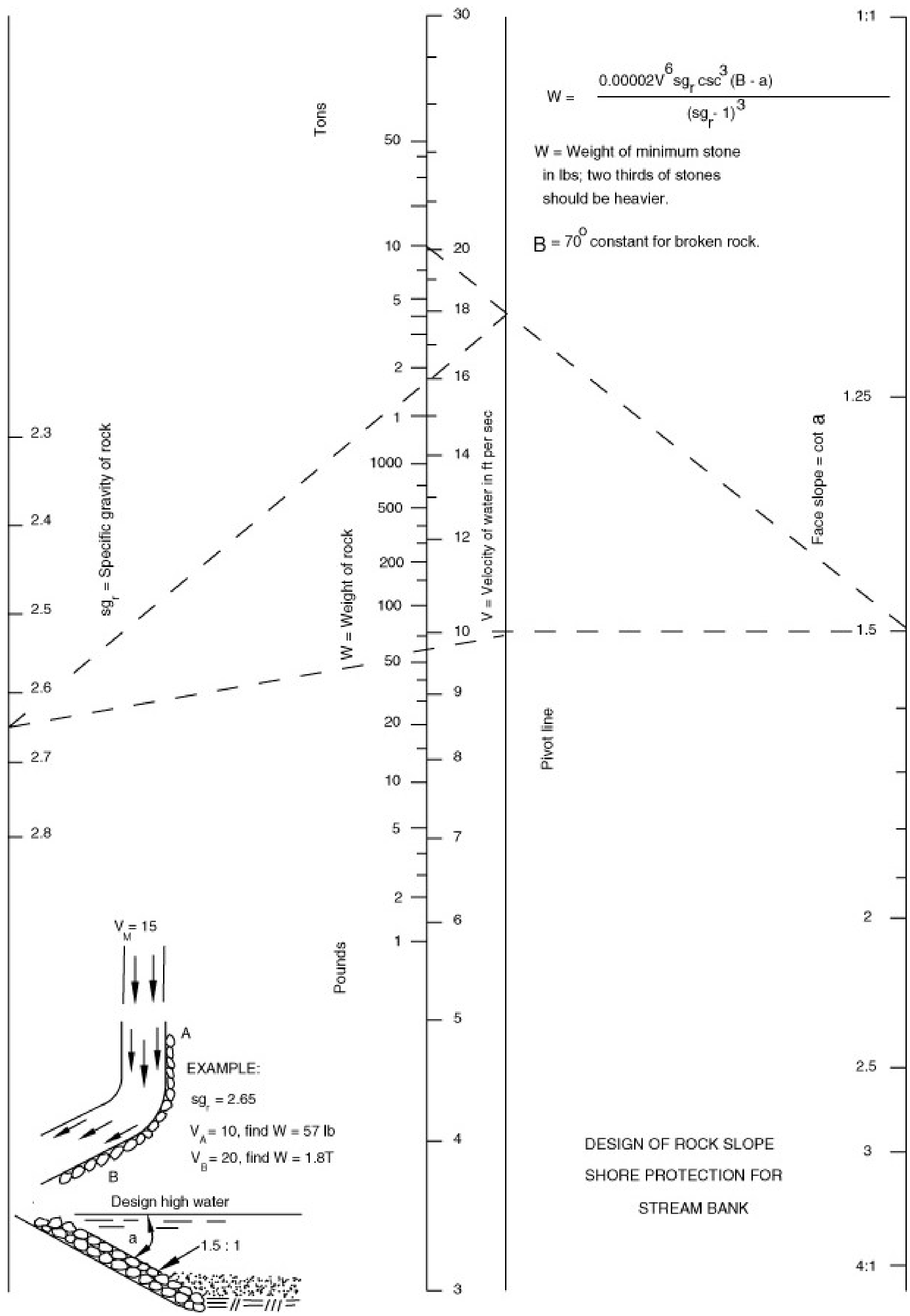
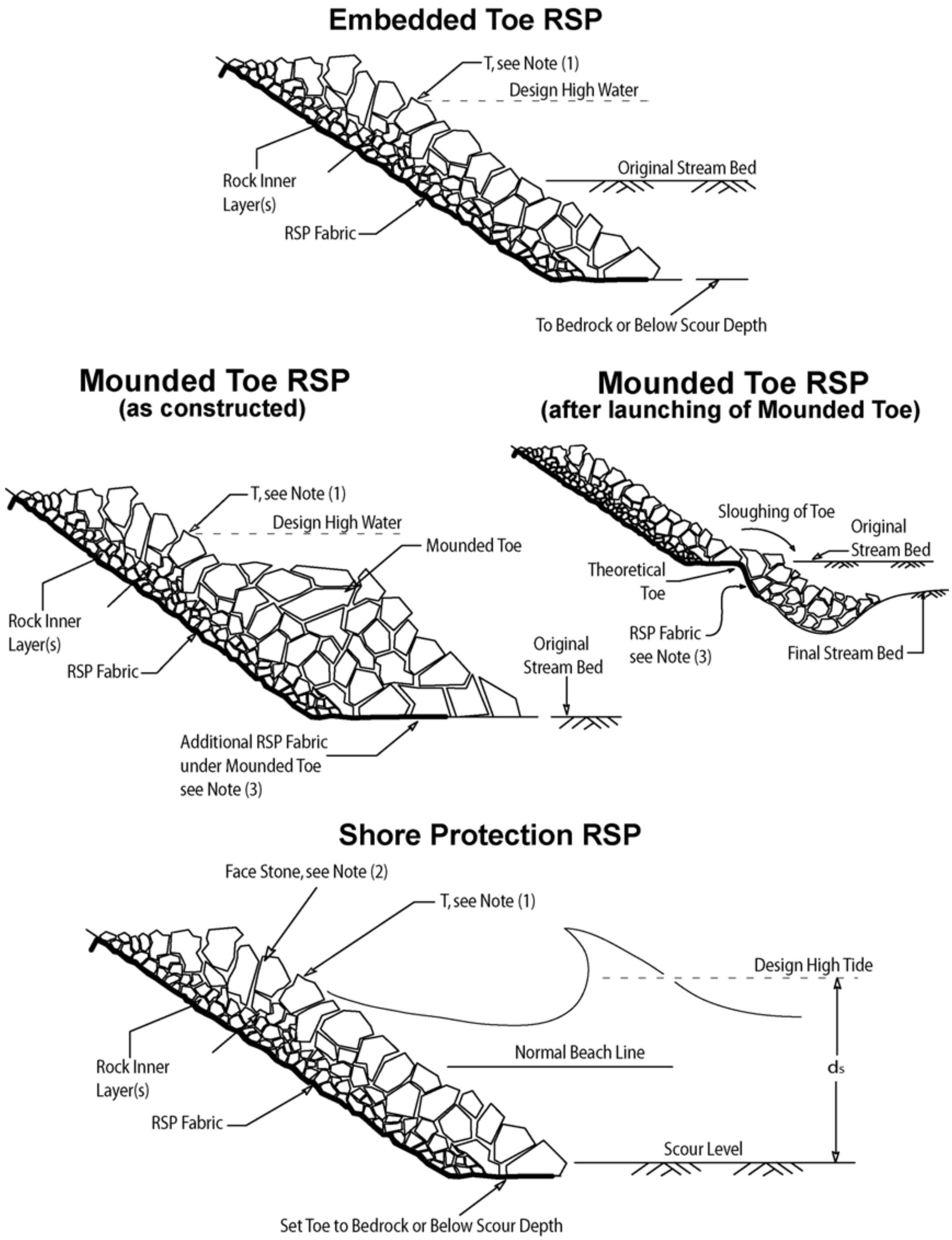


Figure 873.3C
Rock Slope Protection



Notes:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgement.

Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and RSP design considerations.The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event – 16 feet per second
- Estimated scour depth – 5.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 1.5:1
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)\left(16 \times 4/3\right)^6 (2.65)}{(2.65-1)^3 \sin^3(70-33.69)}$$

W = 5,350 lb

W = 2.67 ton = 2.43 tonne

NOTES:

For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

   a) From minimum calculated rock weight of 2.67 tons in the example, select the rock weight from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock weight just larger than the calculated weight. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

Table 873.3A
Guide for Determining RSP-Class of Outside Layer

Standard Rock MASS	GRADING OF ROCK SLOPE PROTECTION PERCENTAGE LARGER THAN											
	RSP-Classes [A]											
	Method A Placement					Method B Placement						
	RSP-Classes other than Backing									Backing No.		
SI Unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1	2	3
16 ton	0-5											
8 ton	50-100	0-5										
4 ton	95-100	50-100	0-5									
2 ton		95-100	50-100	0-5		0-5						
1 ton			95-100	50-100	0-5	50-100	0-5					
1/2 ton				95-100	50-100	-----	50-100	0-5				
1/4 ton					95-100	95-100	-----	50-100	0-5			
200 lb							95-100	-----	50-100	0-5		
75 lb								95-100	-----	50-100	0-5	
25 lb									95-100	90-100	25-75	0-5
5 lb											90-100	25-75
1 lb												90-100

[A] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown.

The next larger rock mass above 2.67 ton is 4 ton. RSP this large is only to be installed using Method A placement techniques (i.e., individual rock placement, no end dumping). From this value, move horizontally across the gradation ranges to the "50-100" entry. From here, move vertically upward to select the design gradation, or RSP Class. In this instance the name of the RSP class is 4 T.

(b) Generally, this will represent the design outer RSP layer. However, the designer must assess this value against the site conditions observed during the field review and in conjunction with site history and projected future conditions prior to finalizing the selection. For the purposes of this example, we will assume this design gradation (i.e., 4 T RSP class) is appropriate.

3) Determine RSP Layers. As previously discussed, properly designed RSP revetments are comprised of multiple layers of progressively smaller rock gradations progressing from the large sized rocks of the outer layer to the native soil or constructed embankment. Where the outer layer is composed of relatively small rock only a single inner layer may be needed. For a large rock outer layer as many as three inner layers may be required.

For this example, the outer RSP layer is 4 T. From Table 873.3B, there are two options for the inner layers. The reason for multiple options for the larger RSP gradation classes is to allow the designer to better select RSP that is available from local quarry sources. Either set of layered designs is acceptable. The designer should contact rock producers in proximity to the project site to obtain price quotes for the different alternatives.

This information may also be available from the District Materials Engineer. For the purposes of this example, we will select the layered design of: 4 T, 1 T, ¼ T, Backing No. 2 and Type B RSP Fabric.

4) Determine Thickness of Revetment. RSP layers are composed of rock classes shown in Table 873.3A. Each layer is at least 1.5 times the diameter of the median sized rock (D_{50}) in the gradation in order to prevent the smaller rocks in the lower layers from migrating.

Table 873.3C provides the required thickness for the various RSP gradations and types of placement (Method A or Method B). Method B placement requires an increase in thickness to account for the looser rock contact and difficulty in controlling layer thickness inherent in end dumping of rock.

Based on the table values, the total thickness of the design in our example (measured normal to the slope) is:

Table 873.3B

California Layered RSP

Outsider Layer RSP-Class *	Inner Layers RSP-Class *	Backing Class No. *	RSP-Fabric Type **
8 T	2 T over ½ T	1	B
8 T	1 T over ¼ T	1 or 2	B
4 T	½ T	1	B
4 T	1 T over ¼ T	1 or 2	B
2 T	½ T	1	B
2 T	¼ T	1 or 2	B
1 T	Light	None	B
1 T	¼ T	1 or 2	B
½ T	None	1	B
¼ T	None	1 or 2	A
Light	None	None	A
Backing No.1 ***	None	None	A

* Rock grading and quality requirements per Section 72-2.02 Materials of the Caltrans Standard Specifications.

** RSP-fabric Type of geotextile and quality requirements per Section 88-1.04 Rock Slope Protection Fabric of the Caltrans Standard Specifications. Type A RSP-fabric has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Type B RSP-fabric.

*** "Facing" RSP-Class has same gradation as Backing No. 1.

Table 873.3 C

Minimum Layer Thickness

RSP-Class Layer	Method of Placement	Minimum Thickness
8 T	A	8.5 ft
4 T	A	6.8 ft
2 T	A	5.4 ft
1 T	A	4.3 ft
½ T	A	3.4 ft
1 T	B	5.4 ft
½ T	B	4.3 ft
¼ T	B	3.3 ft
Light	B	2.5 ft
Facing	B	1.8 ft
Backing No. 1	B	1.8 ft
Backing No. 2	B	1.25 ft
Backing No. 3	B	0.75 ft

4 T Layer = 6.8 ft

1 T Layer = 4.3 ft

¼ T Layer = 3.3 ft

Backing No. 2 Layer = 1.25 ft

RSP Fabric = Effectively

\+ 0.0 ft

Total = 15.35 ft

5) Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed

must be excavated such that the constructed face of the revetment is flush with the original embankment.

6) Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in Index 873.3(2)(a)(2)(b) "Design Height." For depth of toe, the estimated scour was given as 5.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed over the service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 500 feet. Again, this will typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP, see Figure 873.3D for example at ocean shore location.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for ocean shore protection since very large stone is typically needed. Rock mass for lake shores and protected bays are often based on the height of boat generated waves.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production may be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3G.

The nomograph is derived from the following formula:

$$W = \frac{0.003 d_B^{\,3} sg_r \csc^3(\beta - \alpha)}{\left(\dfrac{sg_r}{sg_w} - 1\right)^3}$$

Where:

d_B = maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C

sg_r = specific gravity of stones

sg_w = specific gravity of water (sea water = 1.0265)

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

In general, d_B will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, d_S may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves, see Figure 873.3D, reach the protection, the stone size may be determined by using Chart A, Figure 873.3G. The nomograph is derived from the following formula:

$$W = \frac{0.00231 H_d{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

H_d = design wave in feet, see Index 873.2

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

Figure 873.3D

RSP Lined Ocean Shore

RSP placed at site subject to deep water wave attack. Terminal end of RSP tied into natural rock outcropping.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_S$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_S + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Design thickness of the protection should be based on the same procedures as used for streambanks. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. Shore revetments, particularly ocean shore locations, are often candidates for using a mounded toe design. Where it is not practical to excavate to bedrock or to the anticipated scour depth to set the revetment toe, an alternative treatment is to place additional rock (i.e., mound) of the same mass as the outer layer at the toe. The volume to be placed should be slightly greater than the amount that would have been needed to extend the toe to the estimated scour depth. See figure 873.3C for a depiction of a mounded toe installation.

As scour occurs at the toe of the revetment, this mounded rock will drop into the scour hole. It is important in mounded toe designs to require that excess RSP fabric be placed so that as the scour hole develops and rock begins to drop, the excess RSP fabric will "unfold" and also drop into place to limit loss of embankment.

(c) Gabions. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and Standard Special Provisions 72-300 and 72-305 for specification requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

Refer to the draft Gabion Geotechnical Design Bulletin, available at the following Caltrans Intranet site: http://onramp.dot.ca.gov/hq/esc/sd/bridge_design/ers/documents/gabion_dib.pdf for further discussion on the use of gabions for slope protection.

Figure 873.3E

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

(d) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, and HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(3) *Rigid Revetments.*

(a) Concreted-Rock Slope Protection.

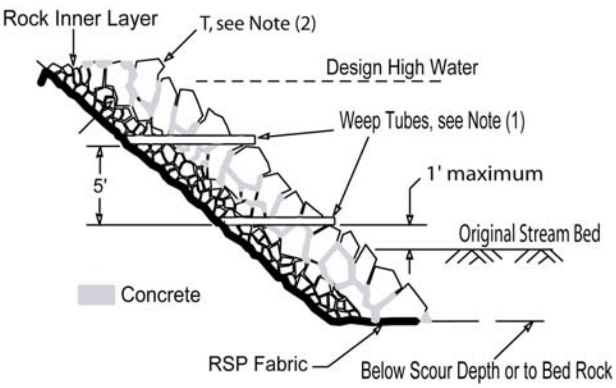
(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3F.

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per

Figure 873.3F

Concreted-Rock Slope Protection



Notes:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

unit area than non-concreted installations.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3H. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

The volume of concrete required is based on filling roughly two-thirds of the void space of the outer rock layer, as shown in Figure 873.3F. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

Figure 873.3G
Nomographs For Design of Rock Slope Shore Protection

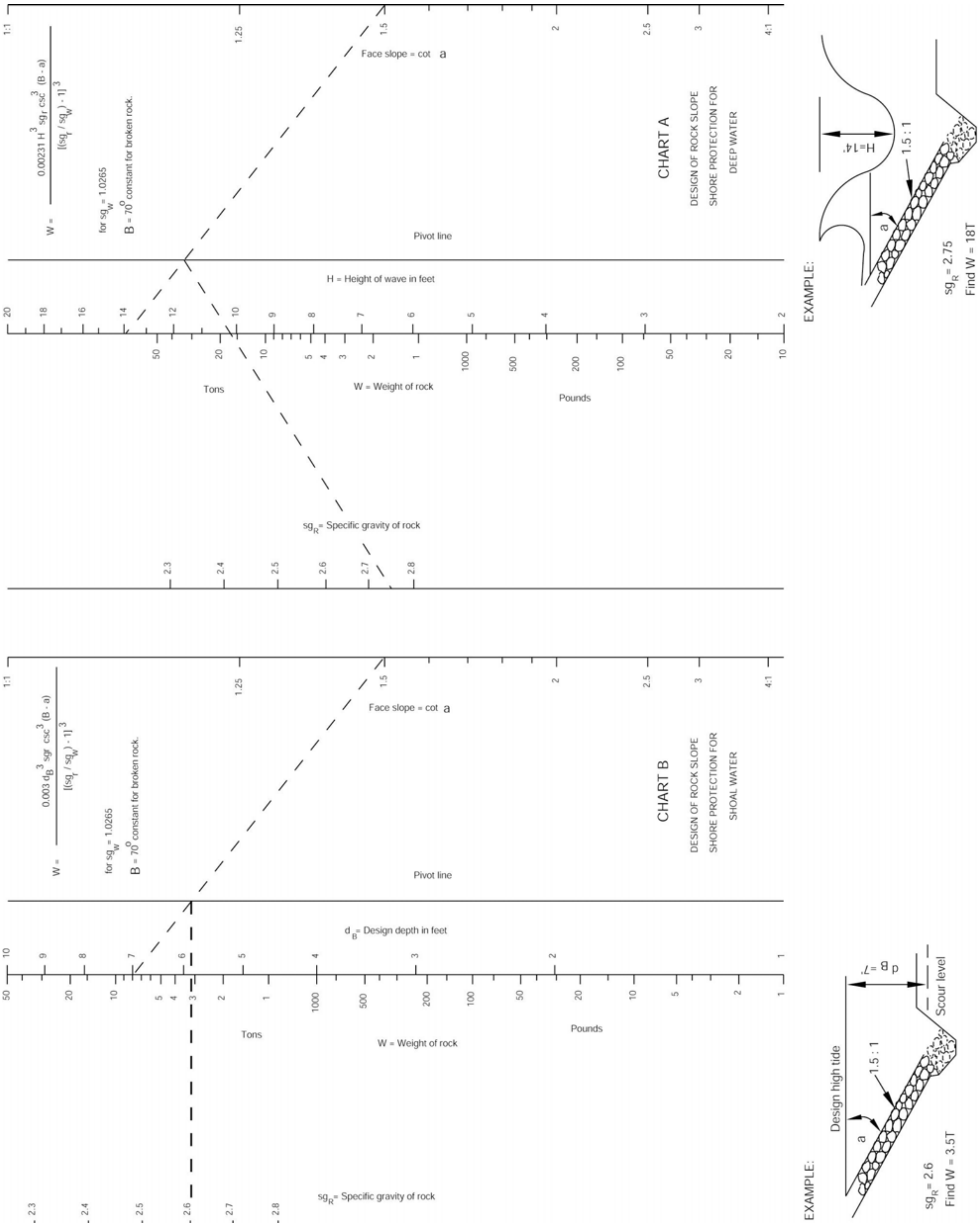


Figure 873.3H

Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

(2) Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate, class 3 or minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specifications section 90, "Portland Cement Concrete."

Size and grading of stone and concrete penetration depth are provided in Standard Specification 72-5.

(b) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Type A RSP fabric as described in Standard Specification Section 88 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3F.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic feet basis the cost is high but as the thickness is generally only 3 inches to 6 inches, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could lead to cracking of the paving or failure. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 1 foot to 2 feet of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving. Table 873.3D gives channel lining thickness.

Table 873.3D

Channel Linings

Mean Velocity (ft/s)	Thickness of Lining (in)		Minimum Reinforcement
	Sides	Bottom	
Portland Cement Concrete or Air Blown Mortar			
< 10	3 – 3.5	3.5 – 4	6 x 6-W2.9 x W2.9 welded wire fabric
10 – 15	4 – 5	5 – 6	#4 Bars at 12 in. and 18 in. centers
15 or more	6 – 8	7 – 8	#3 Bars at 12 in. centers both ways

(4) *Bulkheads.* A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1.5:1 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult the U.S. Army Corps of Engineers EM-1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, for more complete information and details.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7G. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete. Recommendations from METS Corrosion Technology Branch should be requested.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(5) Vegetation. Vegetation is the most natural method for stabilization of embankments and channel bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 862.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 873.3E.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners are:

- Straw
- Excelsior
- Jute
- Woven paper

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Gravel or cobble size riprap
- Fiberglass roving

Table 873.3E

Permissible Velocities for Flexible Channel Linings

Type of Lining [1]	Permissible Velocity (ft/s)	
	Intermittent Flow	Sustained Flow
Vegetation:		
Bermuda Grass, uncut	4.0	2.5
Bermuda Grass, mowed or Crab Grass, uncut	4.0	2.5
Riprap:		
Gravel, 1 in	3.0	2.0
Gravel, 2 in	3.5	2.5
Cobble, 3 in	5.0	4.0
Cobble, 6 in	7.5	6.5
Temporary:		
Woven Paper Net	4.5	3.5
Jute Net	5.0	4.0
Fiberglass Roving	5.5	4.5
Straw with Net	6.5	4.5
Curled Wood Mat	6.5	4.5
Synthetic Mat	10.5	7.5

NOTE:
(1) Ref. HEC-15

- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC-15, Design of Roadside Channels and Flexible Linings.

873.4 Training Systems

(1) General. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 15 feet in streams with sandy beds and velocities of 10 feet per second to 15 feet per second.
- Ends of the system should be joined to the bank in order to prevent parallel high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.
- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated position, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards, refer to Section 6.4.4 of HDS No. 6.

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves. When used in stream environments, a common term used for these devices is spur dike.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. When the purpose is to deposit material and promote growth, the jetties are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A

Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result. See HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 9 for additional information.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4B. They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purpose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

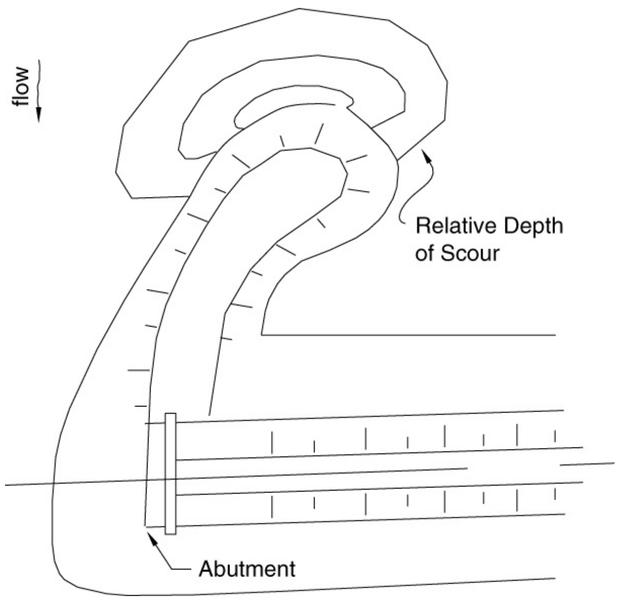
Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependant on the ratio of flow diverted from the flood plain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the flood plain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

For further information on spur dike and guide bank design procedures, refer to Section 6.4 of HDS No. 6. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18, HEC-20, and HEC-23.

Figure 873.4B

Bridge Abutment Guide Banks



(d) Groins. A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

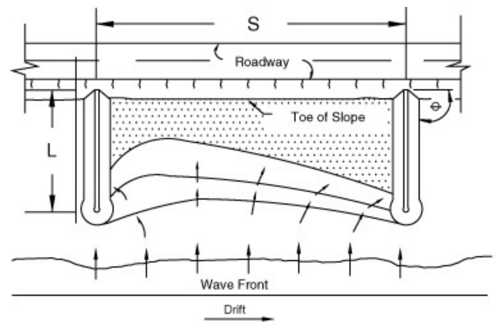
- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

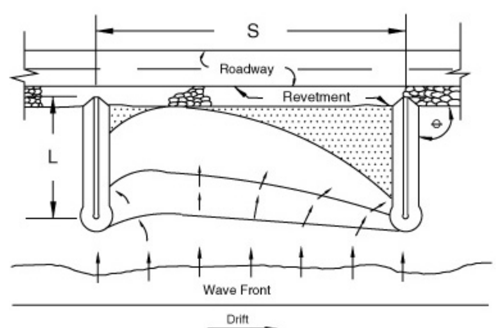
In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 873.4C.

Figure 873.4C

Typical Groin Layout With Resultant Beach Configuration



LONG GROINS WITHOUT REVETMENT



SHORT GROINS WITH LIGHT STONE REVETMENT

Note:
"S", "L" and "θ" are determined by conditions at site.

Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. For a more complete discussion on grouns, designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore Protection Manual until Part VI of the Coastal Engineering Manual is published. Preliminary studies can be made by using basic information and data available from

USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

For a groin to function satisfactorily, there must be littoral drift to supply and replenish the beach between groins. The groins detain rather than retain the drift and soon will be ineffective unless there is a steady source of replenishment. A new groin installation will starve the downcoast beach, temporarily at least, and permanently if the supply of drift is meager. Reference is made to the Army Corps of Engineers' Coastal Engineering Manual, Part III, for more detailed information on the littoral process.

Factors pertinent to design include:

(1) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

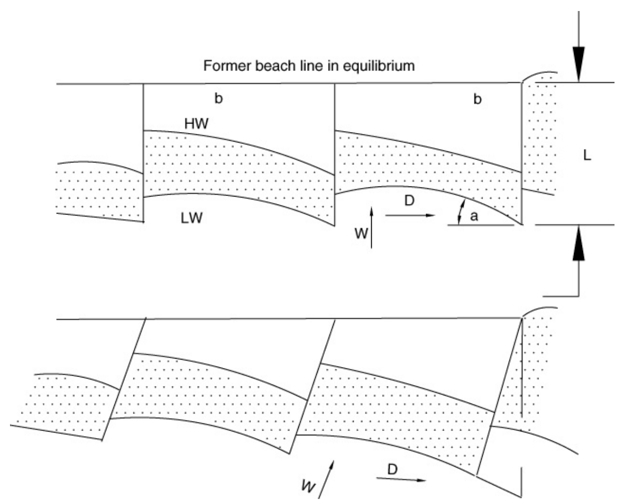
A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 873.4D. The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity, see Figure 873.4D.

Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

Figure 873.4D

Alignment of Groins to an Oblique Sea Warrants Shortening Proportional to Cosine of Obliquity



(2) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 2 feet as a minimum for moderate exposure combined with an abundance of littoral drift, to 5 feet for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct attack of the sea on the end of the groin.

(3) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line, see Figure 873.4D. The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$L = ab + rh$

Where:

L = Length of groin, feet

a = obliquity of entrapped beach in radians

b = beach width between groins, feet

r = reciprocal of beach slope

h = range in stage, feet

For example, with groins 400 feet apart, obliquity up to 20 degrees, on a beach sloping 10:1 with a tidal range of 11 feet,

L = .35 x 400 + 10 x 11 = 250 feet

The same formula would have required L = 390 feet for 800-foot spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and the average cost per foot. For some combination of length and spacing the total cost will be a minimum, which should be sought for economical design.

If groins are too short, the attack of the sea will still reach the highway embankment with only some reduction of energy. Some sites may justify a combination of short groins with light revetment to accommodate this remaining energy.

(4) Section. The typical section of a groin is shown in Figure 873.4E. The stone may be specified as a single class, or by designating classes to be used as bed, core, face and cap stones.

Face stone may be chosen one class below the requirement for revetment by Chart A or B, Figure 873.3G. Full mass stone should be specified for bed stones, for the front face at the outer end of the groin, and for cap stones exposed to overrun. Core stones in wide groins may be smaller.

Width of groin at top should be at least 1.5 times the diameter of cap stones, or wider if necessary for operation of equipment. Side slopes should be 1.5:1 for optimum economy and ordinary stability. If this slope demands heavier stone than is available, side slope can be flattened or the cap and face stones bound together with concrete as shown in Figure 873.3F.

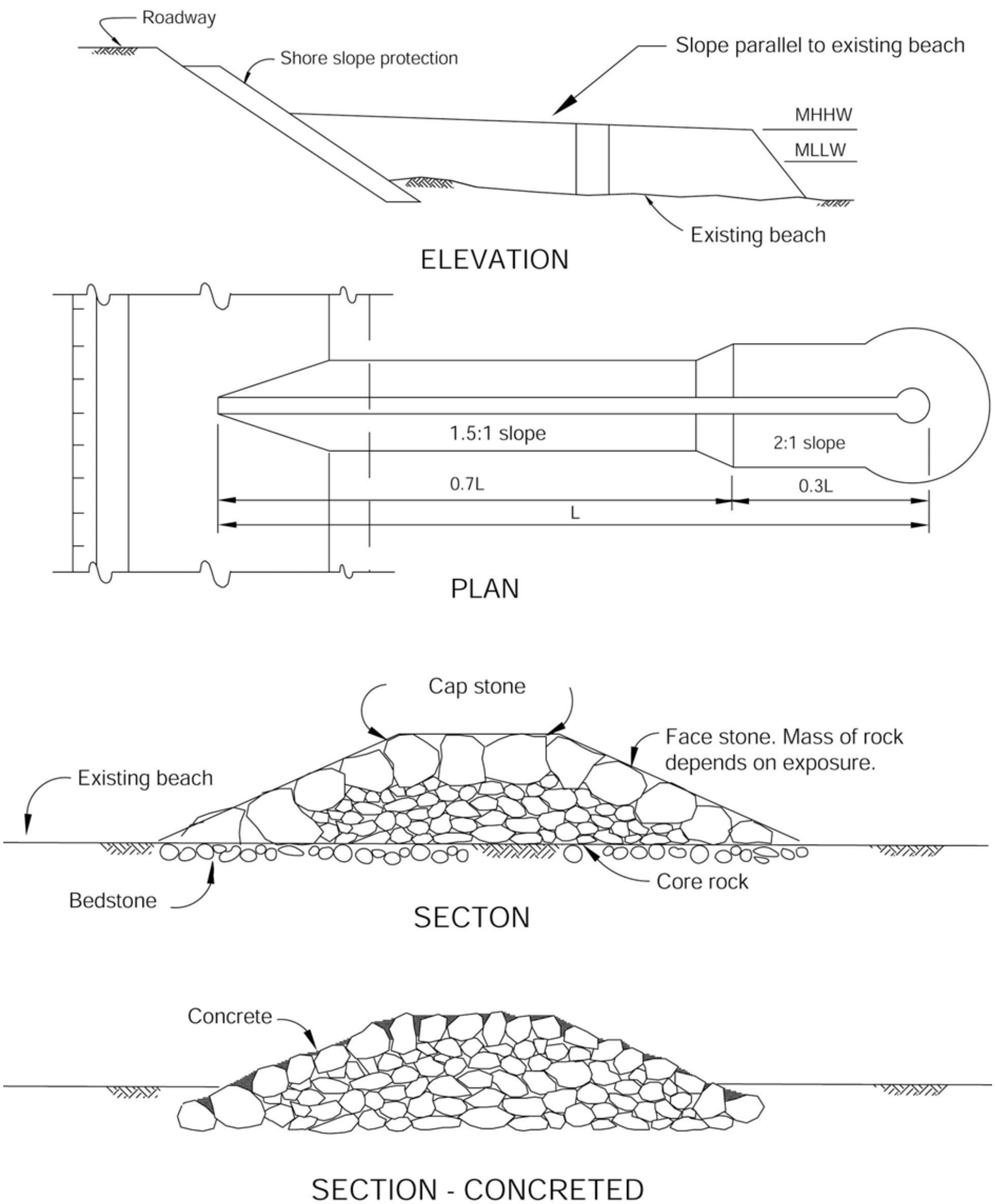
(e) Baffle. A baffle is a pier, vane, sill, fence, all or mound built on the bed of a stream to control, deflect, check or disturb the flow or to float on the surface to dampen wave action.

Baffles typically take the following forms of construction:

- Single or multiple lines of fence.
- Drop Structures (gabions, rock, concrete, etc.).
- Dikes of earth or rock.
- Floating boom.

These devices may vary in magnitude from a check dam on a small stream to a system of training dikes or permeable jetties for deflecting or directing flow. When using fences, palisades, or dikes as deflectors along the more mature valleys or meandering streams, the potential erosion

Figure 873.4E
Typical Stone Dike Groin Details



This is not a standard design.

Dimensions and details should be modified as required.

to previously unexposed areas, threat to adjacent property, eddy currents and possibility of scour should all be assessed. When used as a collecting system to control and direct the flow to new or existing drainage facilities or to bridge openings, the alignment of the installation should be developed as a series of curves and intervening tangents guiding the stream through transitions to maintain smooth and steady flow. The surface and curvature of the training device should be governed by the natural or modified velocity.

Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Refer to HDS No. 6, Section 6.4.11, for further discussion on the use of drop structures.

Floating booms are effective protection against the smaller wave actions common to lakes and tidal basins. Anchorage is the prime structural consideration.

873.5 Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel and shore protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, and sequence of construction should be carefully considered during the project design. The stream and shoreline morphology and their response to construction activities are an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel and shore protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

1. Location of the planned work with respect to:
   - The highway.
   - The stream or shore.
   - Right of way.
2. Datum control of the work, and relation of that datum to gage datum on streams, and both MSL and MLLW on the shore.
3. A typical cross section indicating dimensions, slopes, arrangement and connections.
4. Quantity of materials (per foot, per protection unit, or per job).
5. Relation of the foundation treatment with respect to the existing ground.
6. Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).
7. The limits of excavation and backfill as they may affect measurement and payment.
8. Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.
9. Location and details of construction joints, cut-off stubs and end returns.
10. Restrictions to the placement of reinforcement.
11. Connections and bracing for framing of timber or steel.
12. Splicing details for timber, pipe, rails and structural shapes.
13. Anchorage details, particularly size, type, location, and method of connection.

14. Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.
15. Number and arrangement of cables and details of fastening devices.
16. Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.
17. On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.
18. On fence-type construction the number of lines or rows of fence, spacing of lines, dimensions of posts, details of bracing and anchorage ties, details of ties at end.
19. The details of gabions and the filling material.
20. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.
21. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.